DETERMINATION OF PEAK DISCHARGE AND DESIGN HYDROGRAPHS FOR SMALL WATERSHEDS IN INDIANA

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by

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FOR SMALL WATERSHEDS IN INDIANA

TO: K. B. Woods, Director
Joint Highway Research Project

April 22, 1964

FROM: H. L. Michael, Associate Director
Joint Highway Research Project

Project: C-36-62A
File: 9-8-1

Attached is a Technical Paper entitled "Determination of Peak Discharge and Design Hydrographs for Small Watersheds in Indiana". The paper has been prepared by Mr. I. P. Wu and Professors J. W. Delleur and M. H. Diskin of our staff or formerly of our staff. The paper was presented at the last Annual Purdue Road School and is also intended to be a design manual. The manual has been prepared from research performed at Purdue and in cooperation with the Indiana State Highway Commission and the Indiana Flood Control and Water Resources Commission. Complete details of this cooperation are related in the Preface and Acknowledgement Section of the report.

The attached paper is presented for action as to publication. Since it is intended to be a design manual consideration should be given to separate publication in about the page size of the attached material.

Respectfully submitted,

Harold L. Michael

Harold L. Michael, Secretary

HLM:bc

Attachment

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Determination of Peak Discharge

and

Design Hydrographs for

Small Watersheds in Indiana

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLE

A B S T R A C T

Two simple and practical equations are presented for the determination of the peak discharge of flow from small rural watersheds varying from 5 to 250 square miles. These equations are based on a frequency analysis of peak discharges and a multiple correlation of the 25 year peak discharge to geomorphological characteristics of the watersheds. Peak discharges may be also calculated for return periods of 10, 25, 50, 75, and 100 years. The equations derived are applicable to the design of minor drainage structures such as culverts and small bridges on ungaged streams. Charts which facilitate the design are presented.

A simple method is presented for obtaining a design hydrograph of runoff from small rural basins varying from 3 to 100 square miles. The method is based on the instantaneous unit hydrograph theory. The hydrograph is determined by two parameters which have been statistically correlated to geomorphological characteristics of the watersheds. Practical design charts are presented by means of which the short duration hydrograph and the design hydrograph may be determined from the physiographic conditions and measurements obtained from topographic maps and from the design rainfall. Design rainfall and soil type maps are included for practical application. The method is well suited for the design of small drainage systems and for the hydrologic design of small reservoirs.

The results obtained by the methods must be considered an aid to engineering judgement rather than proven figures. Although in the peak discharge study and in the hydrograph analysis, all the data available from the USGS at the time of these studies were considered, the samples for the statistical analysis were very limited. The estimate of accuracy of the peak discharge and of the hydrograph determination is discussed in the text. The authors hope that in the future, as new data become available, the proposed method may be improved and brought up to date.

PREFACE AND ACKNOWLEDGMENTS

The present manual is based on Report No. 15 of July 1963 submitted to the Joint Highway Research Project, School of Civil Engineering, Purdue University, by I. P. Wu and entitled "Hydrology of Small Watersheds in Indiana and Hydrodynamics of Overland Flow". This study has been sponsored by the State Highway Department of Indiana and by the Indiana Flood Control and Water Resources Commission.

The study was proposed by Dr. J. W. Delleur, Professor of Hydraulic Engineering, Purdue University, at the JHRP Board Meeting of January 24, 1957. The study was started in September 1959 as a JHRP Project, when Mr. I. P. Wu joined the staff of the School of Civil Engineering. A frequency analysis of peak discharges and a method of peak discharge determination were developed between September 1959 and June 1961. The results were presented in progress report No. 1 entitled "Study of Runoff From Small Watersheds for Highway Drainage Design in Indiana", by I. P. Wu and J. W. Delleur, Joint Highway Research Project, May 1961.

The study was extended to include the development of a synthetic hydrograph for small watersheds in Indiana. The hydrograph study was supported by the Indiana Flood Control and Water Resources Commission, where Mr. I. P. Wu worked from July 1961 to September 1962 as a Hydraulic Engineer. The results were published in a paper entitled "Design Hydrographs for Small Watersheds in Indiana" by I. P. Wu, Journal of the Hydraulic Division, American Society of Civil Engineers, November 1963.

The research was continued at Purdue University under the auspices of the Joint Highway Research Project from September 1959 to June 1961. A study was made of the hydrodynamics of overland flow. The final report covering the three phases of the research was submitted to the Board of the JHRP in July 1961. Mr. Wu also utilized the research for the thesis requirement for the Ph.D. Degree entitled "Hydrology of Small Watersheds in Indiana and Hydrodynamics of Overland Flow".

The present manual has been written jointly by Mr. I. P. Wu, Hydraulic Engineer, Indiana Flood Control and Water Resources Commission; Dr. J. W. Delleur, Professor of Hydraulic Engineering and Dr. M. H. Diskin, Visiting Associate Professor of Hydraulic Engineering, both of Purdue University.

The authors are indebted to Mr. [illegible] Brown, former head of the Hydraulic Data Section, Indiana Flood Control and Water Resources Commission, for his suggestions in the development of the theory of synthetic hydrographs. The authors wish to express their thanks to Professors K. B. Woods and H. L. Michael, Director and Associate Director, respectively, of the Joint Highway Research Project, Purdue University, and to Messrs. [illegible], W. F. Becker, and [illegible] Andrews, Chief Engineer, former Technical Engineer, and Head of the Technical Service Division, respectively, of the Indiana Flood Control and Water Resources Commission for their continued cooperation. The authors also wish to thank Mr. C. E. Tate, Hydraulic Engineer, Surface Water Division, USGS, Indianapolis, Indiana for giving access to the flow records used in this study.

1. INTRODUCTION

1.1 Historical Background

The determination of the required waterway area of a bridge, the selection of the size of a culvert or the design of a surface drainage system require an accurate estimate of the peak discharge that is expected to pass through the structure. In the design of many hydraulic structures, the engineer is concerned not only with the maximum discharge but also with the total volume of runoff and its distribution with respect to time—the runoff hydrograph. The routing of a flood through a reservoir to determine the spillway dimensions may be cited as an example in which a knowledge of the hydrograph is necessary.

The peak discharge and the runoff hydrograph are particularly difficult to determine for small watersheds because most of them are ungaged. Particularly in Indiana there is very little information for watersheds less than 200 square miles. At the time of this study ([illegible]) there were only twelve watersheds of less than 100 square miles, and seventeen watersheds with an area between 100 and 200 square miles, for which the USGS was reporting flows.

In view of the lack of data, engineers have used empirical formulas for the determination of the peak discharge and have developed synthetic hydrographs for ungaged watersheds. Many of the existing methods of peak discharge determination fail to take into account the factors upon which the runoff depends. Many of the synthetic hydrographs have been developed for specific locations and are not applicable to Indiana watersheds. The need for better information on hydrology of small watersheds in Indiana was evident.

A research program was initiated at Purdue University to obtain reliable methods, based on all the data available and on the concepts of modern hydrology, for the determination of peak discharges and of hydrographs for ungaged watersheds in Indiana. This report presents a summary of the results of this study, and their application to practical problems. The research included a frequency study of watersheds varying from 20 to 250 square miles; the development of a simple formula and an extended formula for peak discharges for watersheds varying from 50 to 250 square mile, and the development of design hydrographs for watersheds varying from 3 to 100 square miles. The size of the watersheds considered in this study is large enough so that the land use and cover do not affect the peak discharge and the runoff hydrograph in any significant way.

1.2 Available Methods for Peak Discharge Determination

Kinnison (1) in 1946 and Chow (2) in 1962 have given a complete list of empirical formulas which have been proposed in the past for peak discharge determination. The most frequently used formulas are those of Talbot (3) published in 1887, of Meyer (4) published in 1879 and the Rational formula originally derived by Mulvany (5) in 1857. Talbot's formula was originally intended for locations in Illinois. It estimates the waterway area from the watershed area. The formula is:

$$a = CA^{3/4} \qquad (1-1)$$

where a is the required waterway area in square feet, A is the watershed area in acres, and C is a coefficient varying between 1/5 and 1 depending on the slope and character of the watershed. The selection of the coefficient depends, among other things, on the experience of the designer. Due to the various factors that affect the runoff other than the watershed area, the value of the coefficient C cannot be accurately determined to represent all the watershed characteristics. Talbot's formula is

multiple correlation relating it to the watershed area and the precipitation index. This discharge correction is based on a correlation that exists between the error in the [illegible] and the error in the topographic index estimate. The correlation between the [illegible] area and the precipitation index was based on a [illegible] watersheds. This study [illegible] topographic index and [illegible] [illegible] [illegible] [illegible] [illegible]

In 19[illegible] [illegible] [illegible] [illegible] [illegible] watersheds in [illegible]

1-3 [illegible]

The [illegible] unit hydrograph [illegible] [9]. [illegible] representing the direct runoff [illegible]

uniformly over the watershed area at a uniform rate during a given period of time. Since the physical characteristics of the watershed are constant, considerable similarity in the shape of hydrographs may be expected for storms of similar rainfall characteristics.

In 1939, Snyder (10) presented a procedure for the development of synthetic unit hydrographs for ungaged watersheds located in the Appalachian Mountain region. Formulas are given for three elements of the hydrograph: time to peak, peak discharge and the time base. The formulas relate the three elements to watershed characteristics. Knowledge of these three items, combined with the fact that the total depth of runoff must equal one inch, makes it possible to sketch the complete unit hydrograph.

Similar relationships were presented by Taylor and Schwarz (11) in 1952 for the North and Middle Atlantic States.

Edson, (12) in 1951, derived a theoretical expression for the unit hydrograph. He assumed that the runoff is brought to the valley in such a way that the runoff increases with a power x of the time. He also considered that the runoff through the outlet of the watershed decreases exponentially with time. Combining these two influences he found that

$$Q = Ct^{x} e^{-yt} \tag{1-4}$$

in which C is a constant and both x and y are expressible in terms of peak discharge and time to peak.

Dimensionless unit hydrographs, which give the ratio q/q_p of the discharge at any time to the maximum discharge in terms of the ratio t/t_p, where t_p is the time to peak, tend to eliminate the effects of the basin characteristics. Dimensionless unit hydrographs have been proposed by Williams (13), Commons (14) and by the Soil Conservation Service (15).

More recently, the unit hydrograph of infinitesimal duration, that is the hydrograph resulting from an instantaneous rainfall, called the instantaneous unit hydrograph, has received considerable attention. (See section 5-1 for further definition and details). Expressions for the instantaneous unit hydrograph were derived by Nash (16) and by Dooge (17). In deriving these equations, the watershed was assumed to be equivalent to a series of linear reservoirs which are reservoirs for which the storage is proportional to the outflow. It was assumed that the coefficient of proportionality or storage coefficient is the same in all reservoirs. (Fig. 1-1)

The assumption of linearity involved in the unit hydrograph theory have been studied critically by Amorocho and Orlob (18). Recently Diskin (19) studied the non-linearity of the rainfall-runoff process and suggested a description of the watershed in terms of a conceptual model consisting of two parallel chains of reservoirs, each one having a different number of reservoirs and its own storage coefficient.

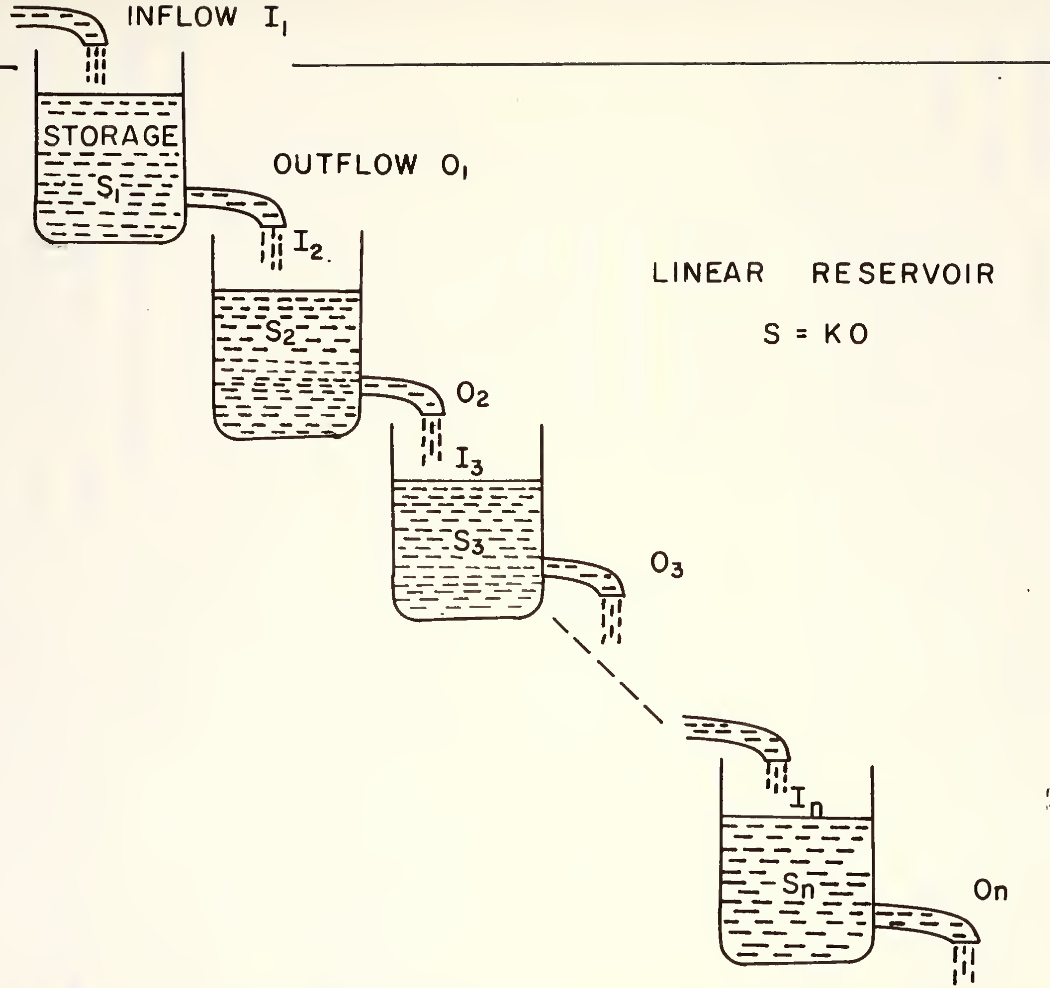


FIG I-I · THEORETICAL MODEL OF A WATERSHED

2. DEFINITIONS AND TERMINOLOGY

2.1 The Physical Characteristics of a Watershed

The watershed, which forms the basic unit considered in this report, is defined with reference to the location of the gaging station or the structure under design. It includes the area within the topographical divide from which water could reach the gaging station or the structure by overland flow. The watershed may be described by a number of properties but for practical purposes only a few of these are usually taken into consideration.

In the present study the following six characteristics were used to describe the watersheds:

(1) Watershed area, A

(2) Main stream length, L

(3) Main stream slope, S

(4) Drainage density, D

(5) Mean relief, H

(6) Watershed shape factor, F

Of these the first three were used for the hydrograph study, characteristics 1 and 3 were used for the simple formula for peak discharge and characteristics 1 and 3 through 6 were used for the extended peak discharge formula. The definition of the watershed characteristics can be done with reference to Fig. 2-1.

1. Watershed area (A) is defined as the area, within the water divide, draining to the gaging station or the structure under design. It is measured from the topographic maps and expressed in square miles.

2. Main stream length (L) is defined as the length measured on a topographic map, along the main stream of the watershed, from the gaging station or from the structure under design upstream to the point where the full blue line on the map ends.

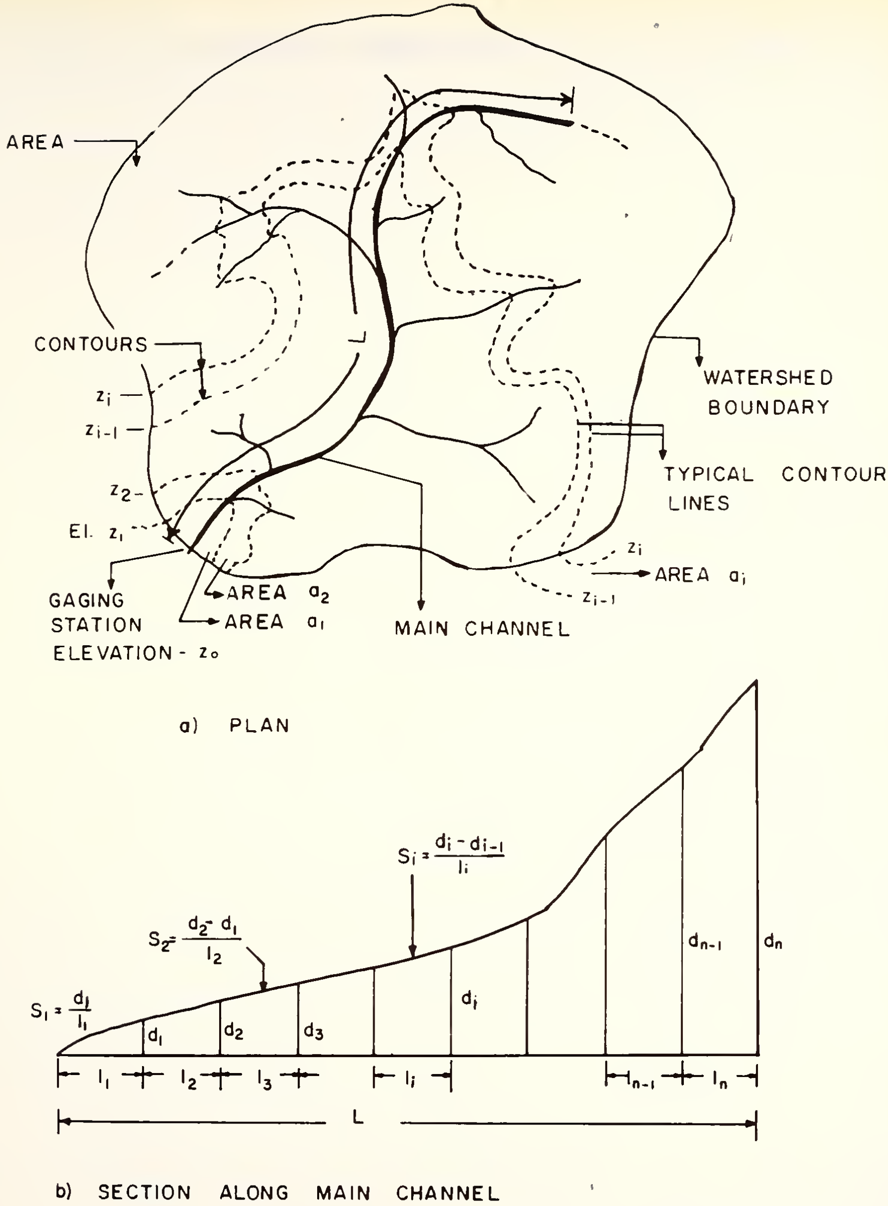


FIG 2-1 DEFINITION OF WATERSHED CHARACTERISTICS

3. Main stream slope (S) is defined with the aid of a longitudinal profile of the main channel. The length L of the main stream is divided into N equal sections and the slope of each section is determined. The main stream slope is then determined by the equation:

$$S = \left[\frac{N}{\frac{1}{\sqrt{S_1}} + \frac{1}{\sqrt{S_2}} + \frac{1}{\sqrt{S_3}} + \cdots + \frac{1}{\sqrt{S_N}}} \right]^2 \qquad (2\text{-}1)$$

where S_1, S_2, S_3 etc. are the slopes of the individual sections. The slope is expressed in feet per 10,000 feet.

4. Drainage density (D) is defined as the ratio of the total length of all streams in the watershed to the area of the watershed. The streams are measured from the drainage maps included in the "Atlas of County Drainage Maps, Indiana" published by Purdue University (20). The drainage density is expressed in miles per square mile.

5. Mean relief (H) is defined as the mean elevation of the watershed above the elevation of the gaging station. If the elevation of the gaging station is z_o and the elevations of the next contour lines are z_1, z_2, z_3,... then the mean relief can be computed by measuring the area within the watershed enclosed by the contour z_1, calling it a_1, and also the areas between the contours z_1 and z_2, between z_2 and z_3 and so on calling the areas a_2, a_3, etc. The mean relief is then given by

$$H = \frac{1}{A} \left(a_1 h_1 + a_2 h_2 + a_3 h_3 + \cdots a_n h_n\right) \qquad (2\text{-}2)$$

$$\text{where } h_1 = \frac{z_1 + z_o}{2} - z_o \; ; \; h_2 = \frac{z_2 + z_1}{2} - z_o \; ; \; h_3 = \frac{z_3 + z_2}{2} - z_o \qquad (2\text{-}3)$$

and n is the number of small areas into which the watershed is divided by the contours. The mean relief is expressed in feet.

6. Watershed shape factor (F) is defined in this study as the ratio of the main stream length to the diameter of a circle having the same area as the watershed. It can be computed by:

$$F = \frac{L}{\sqrt{\frac{4A}{\pi}}} \tag{2-4}$$

2.2 The Total Runoff Hydrograph and Its Components

A runoff hydrograph is by definition a curve showing the discharge at the gaging station as a function of time. The term is used mainly for the portion of the curve obtained during and after a period of rainfall over the watershed. A typical runoff hydrograph for a small watershed is shown in Fig. 2-2. It shows that starting with some low flow in the stream (point A) the discharge rises rapidly to some peak value and then falls gradually to some low value. The two sides of the hydrograph are called the rising curve and the recession curve respectively. The portion of the curve near the peak flow is called the crest section of the hydrograph.

For purposes of analysis the runoff hydrograph is divided into two parts. One part, called the base flow, represents the flow of ground water into the channel system of the watershed; the second part is called the direct surface runoff hydrograph. There are several methods of separating the base flow, but for small watersheds the simplest method was adopted. This method consists of a horizontal line through the point A where the rising curve starts to rise. This horizontal line implies a base flow of constant magnitude Q_B. The total discharge Q_T at any time is then equal to the sum of the base flow Q_B and the direct surface runoff Q

$$Q_T = Q + Q_B \tag{2-5}$$

A curve showing the variation in direct surface runoff Q with time is called the

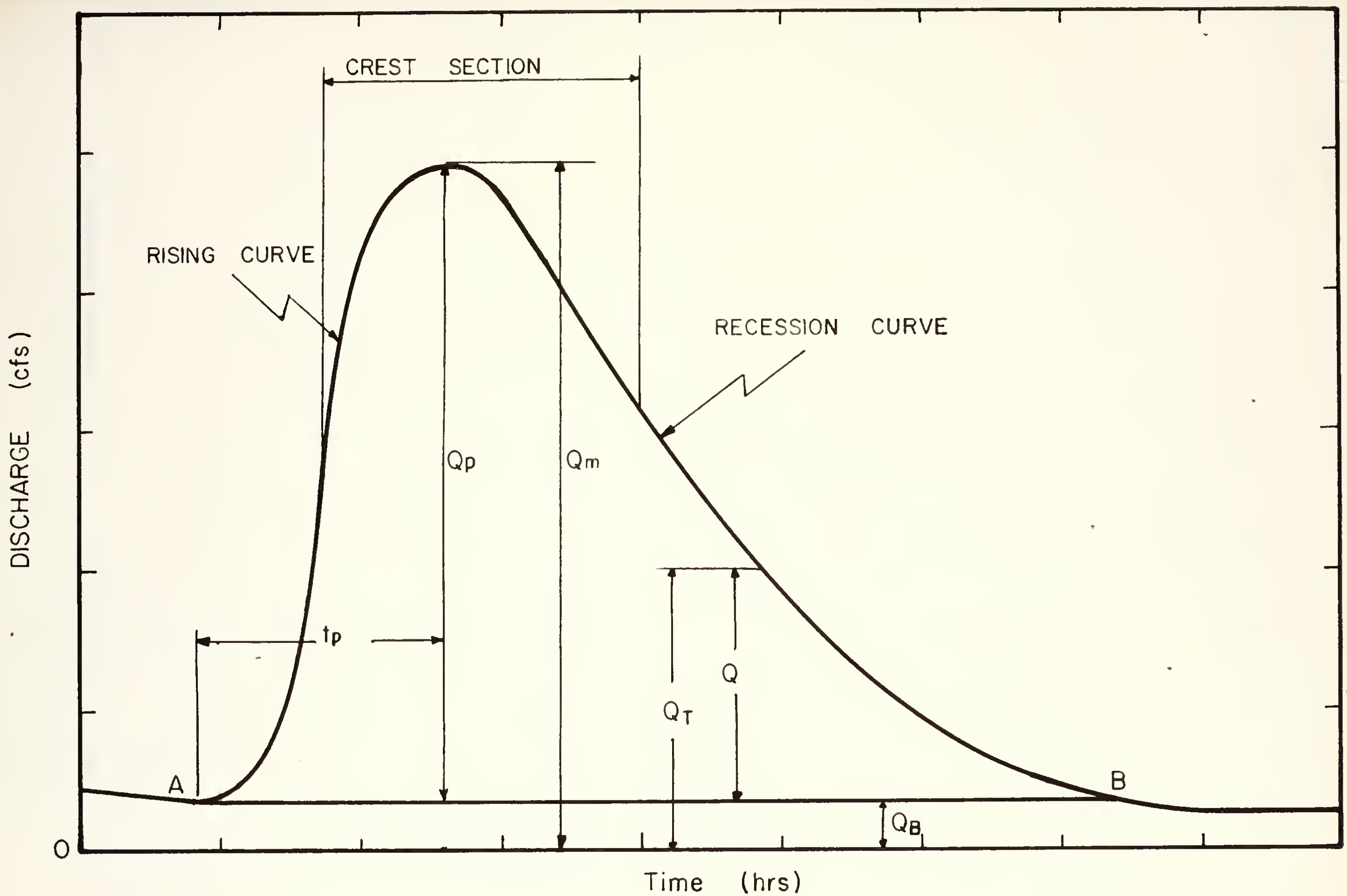


FIG. 2-2 A TYPICAL RUNOFF HYDROGRAPH

direct surface runoff hydrograph (Fig. 2-3). It will be noted that the peak of the direct surface runoff hydrograph (Q_p) is in general smaller than the peak of the corresponding total hydrograph (Q_m), the time to peak (t_p) is the same for the two curves.

The segment of the recession curve of the direct surface runoff hydrograph immediately following the crest section tends to give a straight line when plotted on semi-log paper (discharge on log scale). The equation of such a straight line is

$$\log \frac{Q_o}{Q_1} = \frac{t_1 - t_o}{K_1} \qquad (2\text{-}6)$$

where Q_o and Q_1 are the values of the discharge at times t_c and t_1, and K_1 is called the recession constant of the curve.

The area under the direct surface runoff hydrograph represents the total volume of runoff V which may be expressed in either cubic feet or in units of acre feet. The total volume of runoff is usually considered to be equal to the product of the area of the watershed A and an equivalent depth of water R

$$V = AR \qquad (2\text{-}7)$$

The Quantity R is called the total runoff and is expressed in units of inches. If the area A of the watershed is expressed in square miles and the volume V in acre-feet, equation 2-7 should be modified to include a conversion factor. For the units specified the equation becomes

$$V = \frac{640}{12} AR \qquad (2\text{-}8)$$

2.3 The Total Rainfall Hyetograph and the Rainfall Excess Hyetograph

The rainfall occuring over a watershed is a variable quantity. It varies both with location and with time. For any short period of time (T) it is possible to calculate the mean rainfall over the watershed by standard methods such as the Thiessen polygon method. From the mean rainfall depth it is then possible to derive a mean rainfall intensity for the period under consideration. A

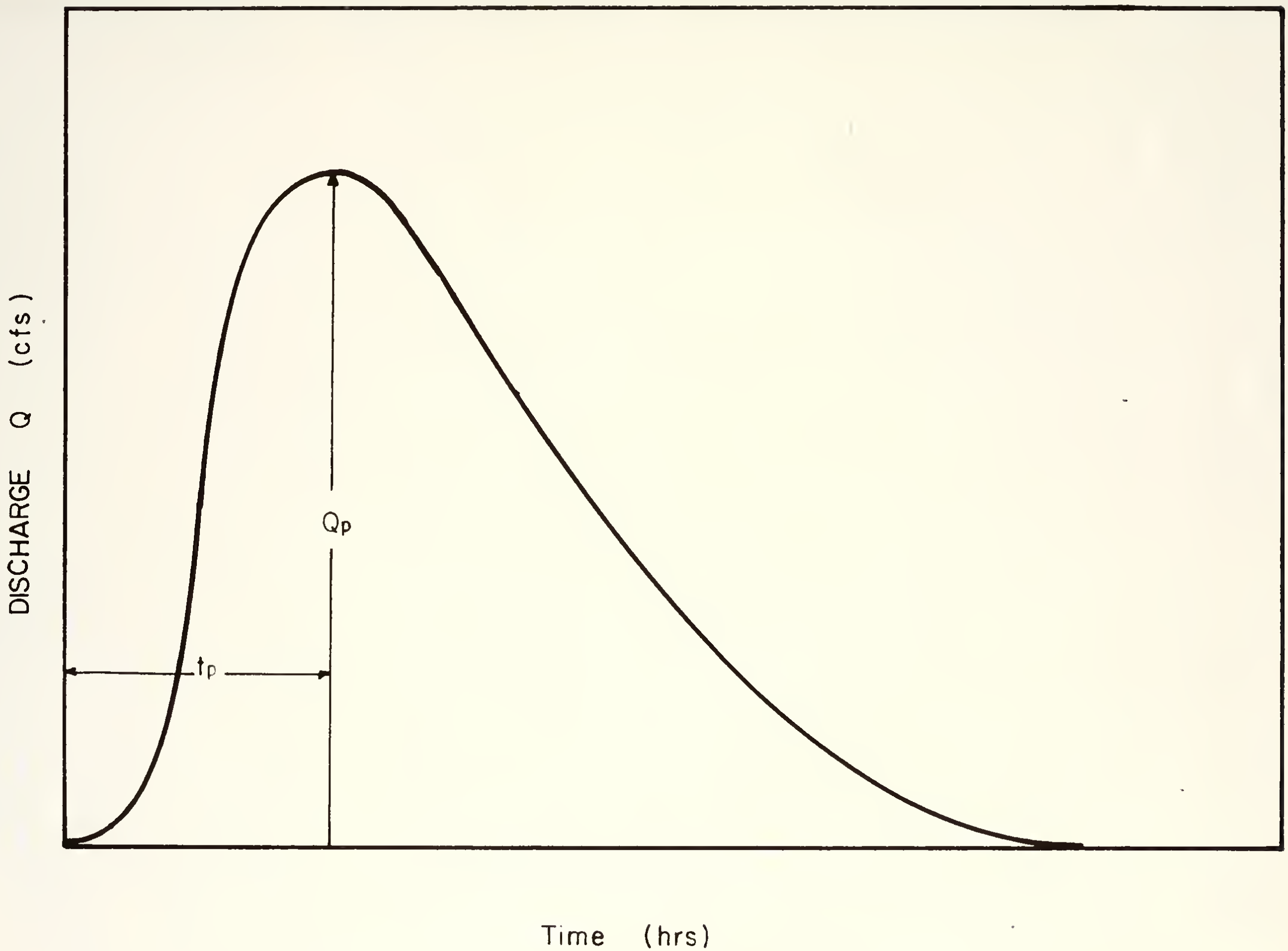


FIG. 2-3 THE DIRECT SURFACE RUNOFF HYDROGRAPH

diagram showing the mean rainfall intensity during successive time periods is called the total rainfall hyetograph for the watershed (Fig. 2-4). It has the shape of a bar diagram and the property that the area under each of the bars is equal to the rainfall depth during the corresponding time interval.

The total area under the hyetograph is equal to the (mean) total precipitation depth P over the watershed during the storm, it is expressed in units of inches. Comparing the value of P with the value of the total runoff R for the same storm, it is found that almost invariably the total rainfall P is larger than the total runoff R. For purposes of analysis it is usual to divide the total rainfall hyetograph into two parts. One part represents the portion of the rainfall that appears as runoff at the gaging station and the second represents the rainfall lost through infiltration, evapotranspiration, and other causes. A procedure for separating the two parts, suitable for small watersheds is the following:

(a) By examining the runoff hydrograph the time of beginning of direct surface runoff (point A in Fig. 2-2) is found. All rainfall before this time is considered to be an initial loss. The depth of rainfall included in this initial loss is represented by area under the hyetograph up to this time. If the depth of initial loss is denoted by P_L and the depth of precipitation after the beginning of direct surface runoff by P_x then the total precipitation is given by

$$P = P_L + P_x \qquad (2\text{-}9)$$

(b) For the portion of the total hyetograph after the beginning of direct surface runoff a horizontal line is found by trial and error such that the depth of rainfall represented by the portion of the diagram above the line is exactly equal to the total runoff R. The line is called the separation line for rainfall excess and the portion of the total hyetograph above the line is called the rainfall excess hyetograph (Fig. 2-5). The ratio of the total runoff R to the depth of precipitation P_x was defined in this report as the runoff coefficient r

$$r = R/P \qquad (2\text{-}10)$$

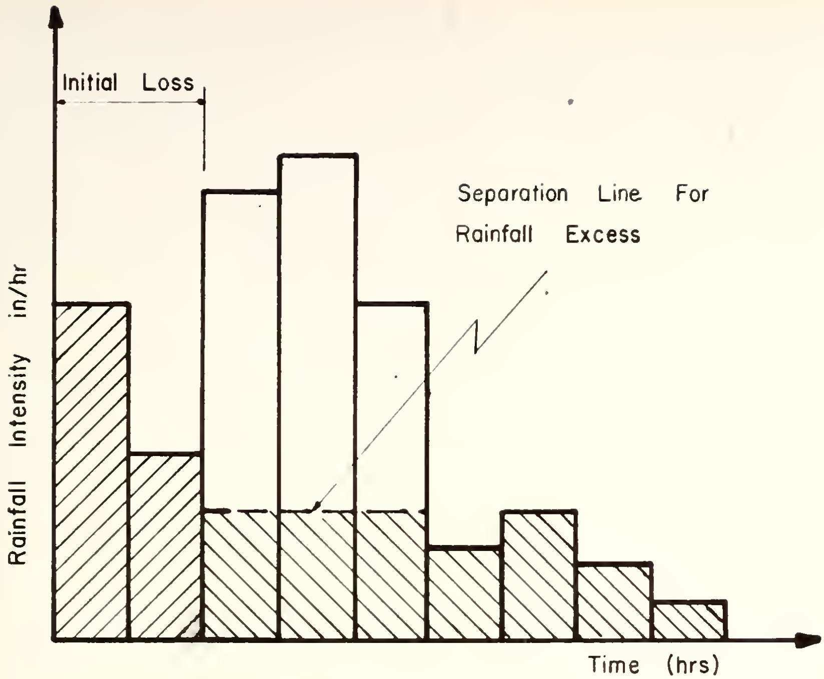


FIG. 2-4 THE TOTAL RAINFALL HYETOGRAPH

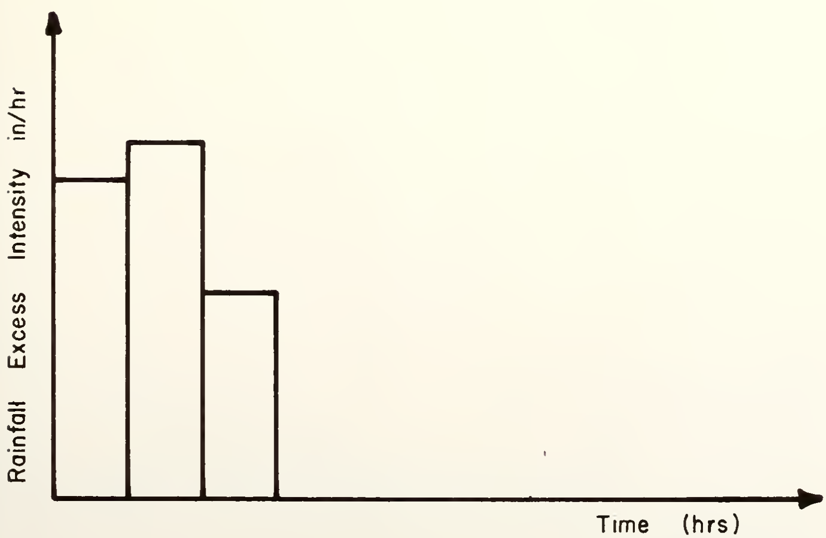


FIG. 2-5 THE RAINFALL EXCESS HYETOGRAPH

2.4 Unit Hydrographs of Short Duration

The unit hydrograph forms a convenient basis for comparison of the direct surface runoff hydrographs of a watershed. By definition, the unit hydrograph is a direct runoff hydrograph of unit total runoff, in this case one inch of direct runoff. The unit hydrograph is derived from the observed direct surface runoff hydrograph by dividing the ordinates of the latter curve by the total runoff R.

Each unit hydrograph is associated with the duration of the rainfall excess which produced it. Thus, a 3-hour unit hydrograph is one derived from a storm in which the duration of the rainfall excess was 3 hours. Using the assumption of linear relationship between rainfall and runoff, it is possible to derive a unit hydrograph of any one duration from a unit hydrograph of any other duration by superposition or by using the S-curve technique.

The shape of the unit hydrograph depends on its duration, as this duration becomes smaller the shape tends towards some limiting form. The instanteneous unit hydrograph, which is the limiting form of the unit hydrographs as the duration becomes infinitesimally small is useful in theoretical studies but its derivation requires special techniques. For practical purposes, a unit hydrograph derived from hydrographs due to rainfalls of short duration, of the order of 0.1 t_p, may be used as an approximation of the instantaneous unit hydrograph of the watersheds considered. Such a unit hydrograph can be derived from past records by selecting a number of hydrographs with high and sharp peaks, short time to peak, and smooth recession curves, reducing them to a dimensionless form and passing an average curve through the dimensionless curves plotted on a common basis.

The dimensionless form used in the report for the unit hydrograph of short duration is obtained by expressing the flow as a ratio of the peak flow (Q/Q_p) and the time as a ratio of the time to peak (t/t_p).

By assigning a definite but short duration to the instantaneous unit hydrograph it is possible to derive unit hydrographs of longer durations, or runoff hydrographs, by a numerical procedure.

2.5 Definition of Statistical Terms

Two statistical techniques were utilized in this study. One is an extreme value analysis and the second is that of multiple correlation analysis.

The extreme value analysis is based on the principle that if the largest (or smallest) value is chosen from each of a number of samples of a variable, the series of extreme values so chosen will have a statistical distribution which will be independent of the statistical distribution of the original samples from which the extreme values are taken. It can be shown ([illegible]) that the above statement is correct if the number of individual items in each of the samples is large and if the number of samples from which the extremes are taken is also large. The theoretical distribution of the extreme values series can be expressed mathematically ([illegible]). For practical application a special probability curve for the extreme values series appears as a straight line on this probability paper. The probability paper has, along one of the axes, a cumulative probability scale $\phi(x)$, and in addition also a parallel scale marked in units of the return period T_r in years. The relationship between the two scales is given by

$$T_r = \frac{1}{1 - \phi(x)} \qquad (2\text{-}10)$$

The second axis is scaled in units of the extreme values plotted and is denoted as the X-axis.

Instead of computing the probability for each of the entries in the extreme values series it is simpler to compute the return period for each of the entries and use the auxiliary scale on the probability paper. To do this, the entries are arranged in a decreasing order of magnitude and assigned rank numbers (m) according to their relative position, thus m=1 for the largest value, m=2 for the

second largest and so on. The return period for each entry is then calculated by

$$T_r = \frac{n + 1}{m} \qquad (2\text{-}11)$$

where n is the total number of entries in the extreme value series.

The extreme value analysis and special probability paper were used in this study for the analysis of the annual peak flows and for the prediction of the 25-year peak flow, which was used for the correlation with watershed characteristics. The entries in the extreme values series were, in this case, the instantaneous peak discharge measured at the gaging station for each of the years in the period of record.

The multiple correlation analysis is a technique for deriving the parameters of the equation relating a number of variables. It is based on the observed values of the variables, and is designed to give the values of the parameters which will make the sum of squares, of the deviations of the computed values of the dependent variable from the observed values, obtain a minimum value. The equations used in this study to express the relationship between the variables was of the type

$$y = C\, x^a z^b s^c t^d \qquad (2\text{-}12)$$

in which x, z, s, t are the independent variables and y is the dependent variable. C, a, b, c, d are the parameters of the equation, the values of which are determined by the correlation analysis to give a minimum value of the sum of squares. In the application of the correlation analysis to the type of equations given by Equation 2, they are transformed first into a simpler form by taking logarithms of their terms

$$\log y = \log C + a \log x + b \log z + c \log s + d \log t \qquad (2\text{-}13)$$

and then forming a set of simultaneous equations in which a, b, c, d and (log C) are the unknown quantities. The solution of this set of simultaneous equations gives the values of the unknowns which will make the sum of the squares of the

deviations of the computed (log y) values from the observed (log y) values the smallest possible with the observed data.

3. DESCRIPTION OF THE BASIC DATA

3.1 Watersheds Studied

Forty-two watersheds distributed throughout the state of Indiana were selected for the studies of peak discharge and for the hydrograph determinations. Fig. 3-1 is a map showing the location of the watersheds and Table 3-1 lists the names of the watersheds, their assigned numbers and their areas. Table 3-1 also indicates which of the watersheds were used for the various studies included in this report. Thirty-two watersheds were used for the frequency study, sixteen for the peak discharge correlation and seventeen for the hydrograph study.

3.2 Watersheds and Records for Peak Discharge Determination

The thirty-two watersheds selected for the frequency study are indicated in Table 3-1 with a star (*) in column "a". A bar diagram was plotted in Fig. 3-2 to show the time period of record for each of these 32 watersheds. The instantaneous annual peak flows, used in the frequency study, were obtained from the report by Green and Hoggatt (23) and from data supplied by the U.S.G.S. office in Indianapolis, Indiana.

At the time of this study (1960), topographic maps were available for only sixteen of these watersheds; they are indicated in Table 3-1 with a star in column "b". The multiple correlation formula for peak discharge determination was based on data obtained from the topographic maps for these 16 watersheds. The following properties were measured from the available topographic maps:

(1) watershed area, A
(2) main stream slope, S
(3) mean relief, H
(4) watershed shape factor, F

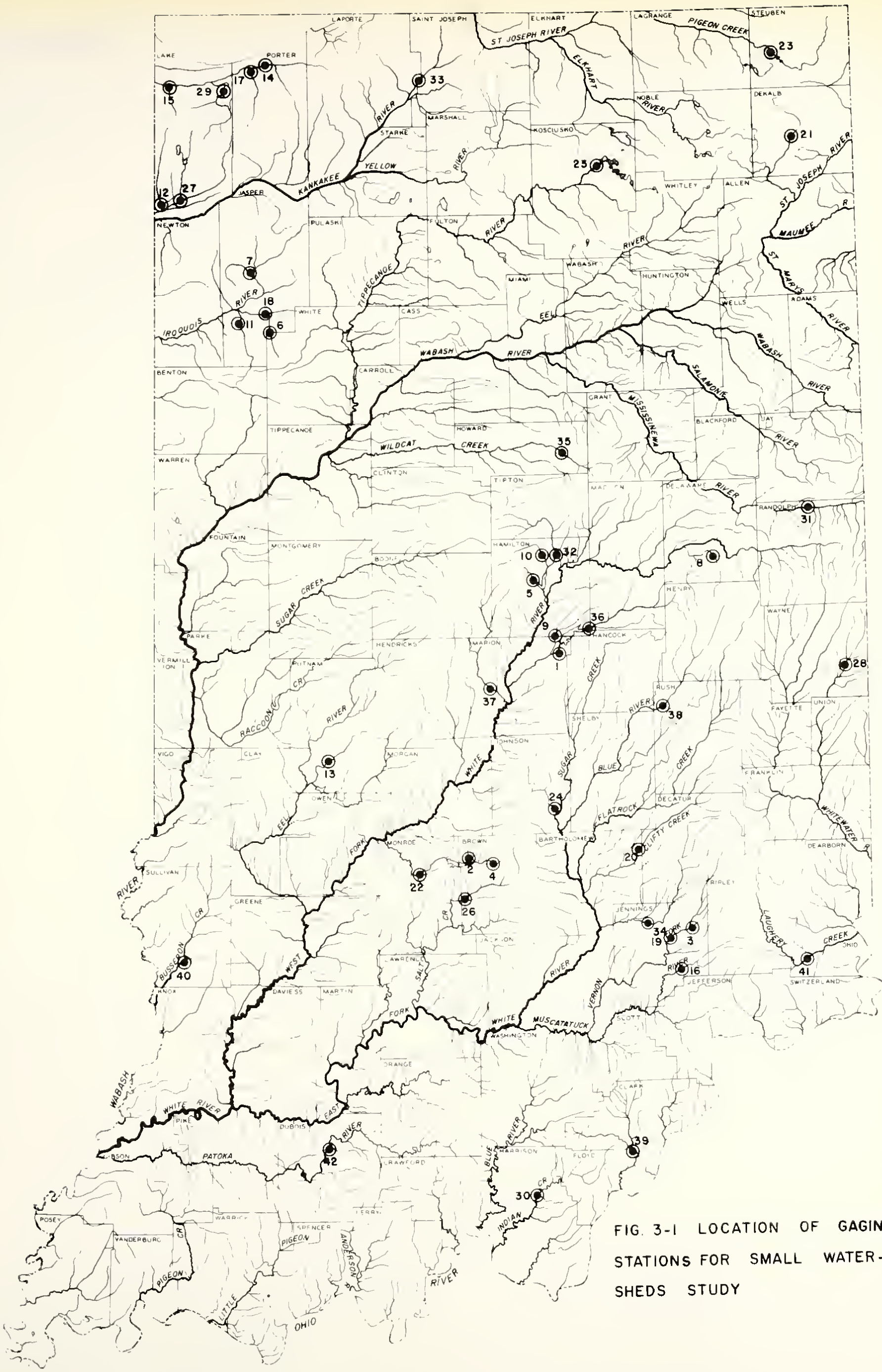
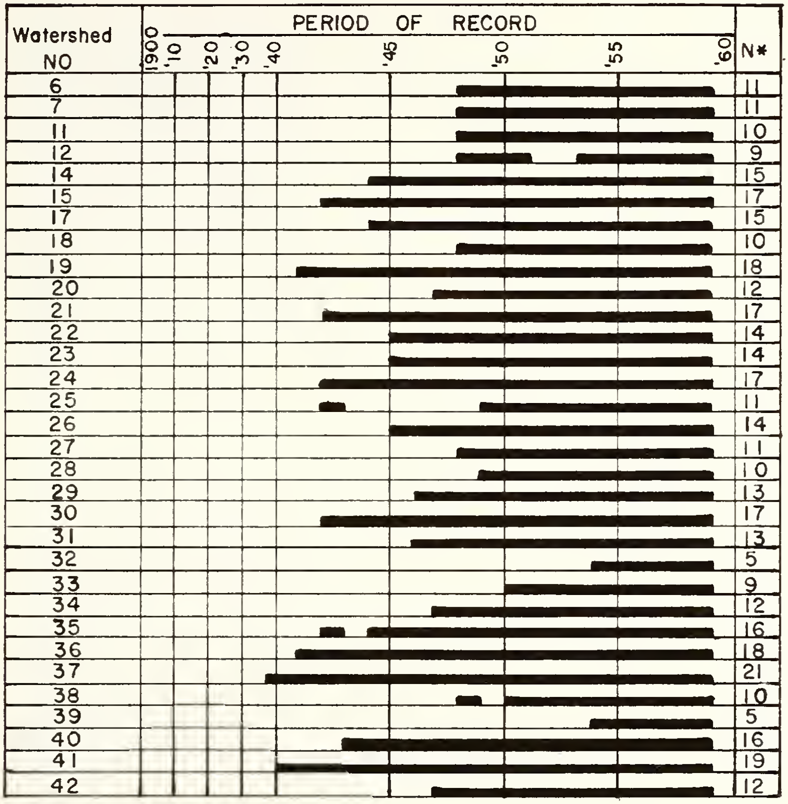


FIG. 3-1 LOCATION OF GAGING STATIONS FOR SMALL WATER-SHEDS STUDY

FIG. 3-2 PERIOD OF RECORD OF INSTANTANEOUS ANNUAL PEAK DISCHARGE USED IN FREQUENCY STUDY



* N = Length of Record in Years

Table 3-1

List of Watersheds, their Area and Assigned Number

Watershed Number	Gaging Station	a	b	c	Watershed Area, A (sq. mi.)
1	Lawrence Creek at Fort Benjamin Harrison			*	2.86
2	Bear Creek near Trevalac			*	7.0
3	Brush Creek near Nebraska			*	11.7
4	Bean Blossom Creek at Bean Blossom			*	14.6
5	Hinkle Creek near Cicero			*	16.3
6	Bice Ditch near South Marion	*			22.6
7	Iroquois River at Rosebud	*			30.3
8	Buck Creek near Muncie			*	36.7
9	Mud Creek at Indianapolis			*	42.5
10	Little Cicero Creek near Arcadia	*		*	44.7
11	Carpenter Creek at Egypt	*			48.1
12	West Creek near Schneider	*		*	54.3
13	Deer Creek near Putnamville			*	59.0
14	Little Calumet River at Porter	*	*	*	62.9
15	Hart Ditch at Munster	*			69.2
16	Graham Creek near Vernon			*	77.6
17	Salt Creek near McCool	*	*	*	78.7
18	Big Slough Creek near Collegeville	*			84.1
19	North Fork Vernon Fork near Butlerville	*		*	87.3
20	Clifty Creek at Hartsville	*	*	*	88.8
21	Cedar Creek at Auburn	*	*	*	93.0
22	Bean Blossom Creek at Dolan	*	*	*	100.0
23	Pigeon Creek at Hogback Lake Cutlet near Angola	*	*		102
24	Young Creek near Edinburg	*	*		109
25	Tippecanoe River at Oswego	*	*		115
26	North Fork Salt Creek near Belmont	*	*		120
27	Singleton Ditch at Schneider	*			122
28	East Fork White Water River at Richmond	*			123
29	Deep River at Lake George Outlet at Hobart	*	*		125
30	Big Indian Creek near Corydon	*	*		129

Continued

a Watersheds used for frequency study
b Watersheds used for peak discharge study
c Watersheds used for hydrograph study

Table 3-1 (Continued)

List of Watersheds, their area and assigned Number

Watershed Number	Gaging Station	a	b	c	Watershed Area, A (sq. mi.)
31	Mississinewa River near Ridgeville	*			130
32	Cicero Creek near Arcadia				131
33	Kankakee River near North Liberty	*			152
34	Sand Creek near Brewersville	*	*		156
35	Wildcat Creek at Greentown	*			162
36	Fall Creek near Forville	*			172
37	Eagle Creek at Indianapolis	*	*		179
38	Blue River at Carthage	*			187
39	Silver Creek near Sellersburg	*	*		188
40	Busseron Creek near Carlisle	*	*		228
41	Laughery Creek near Farmers Retreat	*			248
42	Patoka River at Jasper	*	*		257

a Watersheds used for frequency study
b Watersheds used for peak discharge study
c Watersheds used for hydrograph study

In addition, the drainage density (D) was measured for the same watersheds from the drainage maps. (20) The values determined are listed in Table 3-2. (For definition of the physical characteristics of the watersheds see Art. 2.1)

3.3 Watersheds and Records for Hydrograph Study

The seventeen watersheds selected for the hydrograph study are indicated in Table 3-1 with a star in column "c". Five to six hydrographs for each of the 17 watersheds were selected and used for the determination of the hydrograph parameters. The runoff hydrographs were obtained from the U.S.G.S. office in Indianapolis, Indiana.

For the watersheds used for the hydrograph study, the following characteristics were measured from the available topographic maps:

(1) watershed area, A
(2) length of main stream, L
(3) slope of main stream, S

The values determined are listed in Table 3-3. (For definition of the physical characteristics of the watersheds see Art. 2.1)

3.4 Rainfall Records and Rainfall Characteristics for Indiana

The rainfall records used for the runoff coefficient study were obtained from the publication of the U.S. Weather Bureau, entitled "Climatological Data, Indiana".

Rainfall data for prediction of design storms is available from the Weather Bureau. Recent (1961) data on rainfall-depth-duration-frequency relations can be found in Technical paper No. 40, (24) published by the Weather Bureau. Figures 3-3 and 3-4, which are based on this technical report, show the six-hour duration rainfall for return periods of 25 and 50 years.

A list of ratios to convert the six-hour duration rainfall to rainfalls at other durations, which was prepared by the Soil Conservation Service, is given in Table 3-4.

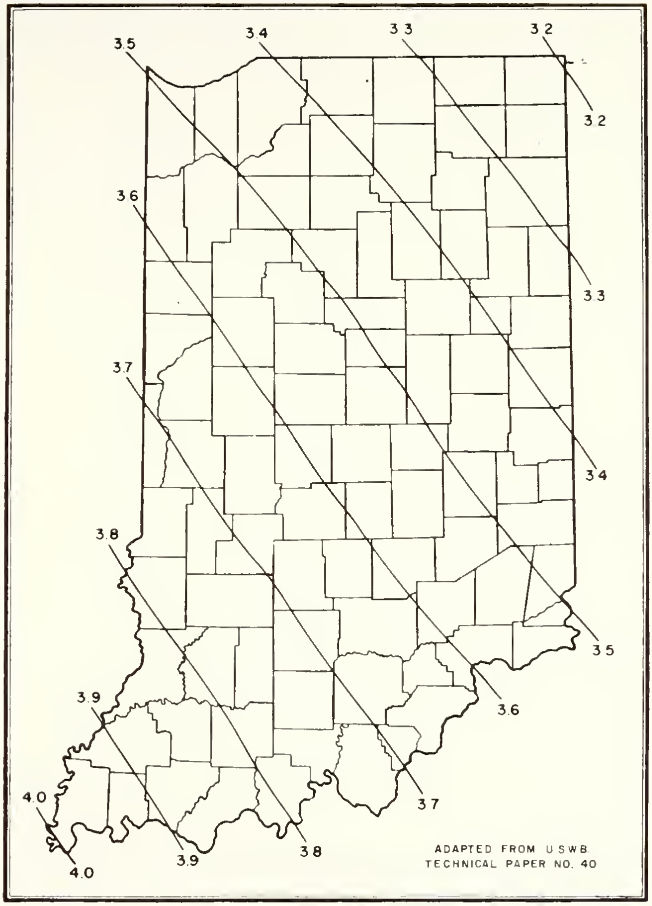


FIG. 3-3 25-YEAR, SIX HOUR RAINFALL IN INCHES FOR INDIANA

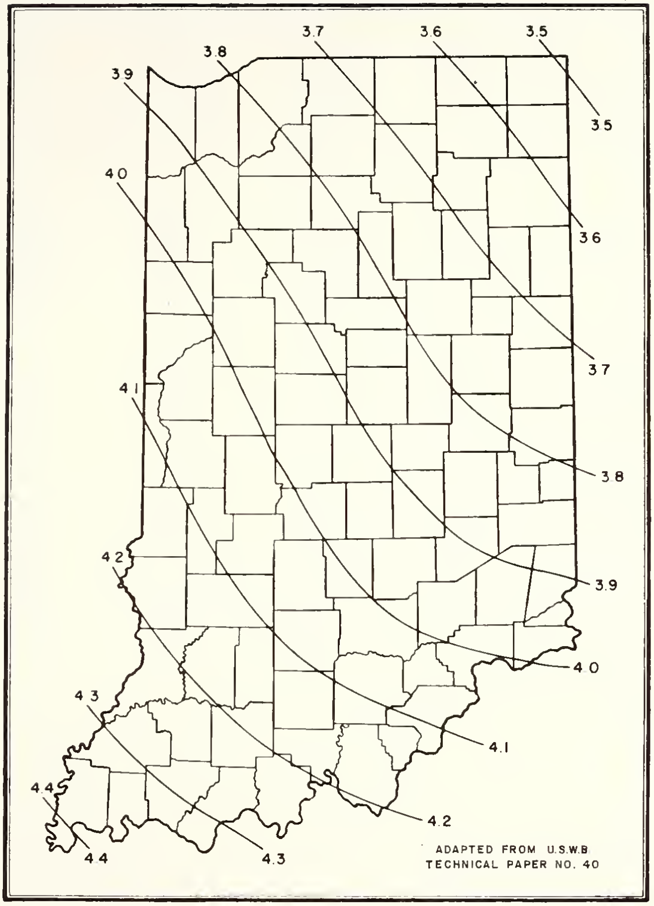


FIG. 3-4 50-YEAR, SIX HOUR RAINFALL IN INCHES FOR INDIANA

Table 3-2

Watershed characteristics of 16 Watersheds used for peak discharge study

Watershed number	Watershed Characteristics				
	Area A (sq. mi.)	Mean Relief H. (ft)	Drainage Density D (mi. sq. mi.)	Shape Factor F	Main Stream Slope S ft/ 1000 ft
14	62.9	110	8.00	1.12	21.10
17	78.7	101	6.57	1.75	9.05
20	88.8	270	7.38	3.00	20.88
21	93.0	79	5.10	1.47	8.29
22	100	216	10.66	2.63	9.84
23	102	66.1	3.16	1.93	7.93
24	109	86	7.02	1.94	10.39
25	115	65.4	3.35	1.41	2.64
26	120	237	11.20	2.18	9.90
29	125	84.7	4.50	1.91	6.05
30	129	231	8.70	2.56	10.16
34	156	250	9.75	2.85	10.68
37	179	195.2	7.88	2.2	13.40
39	188	195.8	10.47	1.35	6.21
40	228	99.8	13.20	1.93	5.43
42	257	181.5	13.95	2.87	2.95

Table 3-3

Watershed Characteristics of 17 Watershed used for hydrograph study

Watershed number	Area A (sq. mi)	Length of main stream L (mi.)	Slope of main stream S (ft/1000 ft.)
1	2.86	1.82	103.00
2	7.0	4.29	63.50
3	11.7	7.28	44.00
4	14.6	7.05	32.60
5	16.3	7.15	20.00
8	36.7	12.25	16.00
9	42.5	18.25	12.00
10	44.7	14.76	12.00
12	54.3	20.50	5.00
13	59.0	17.00	25.50
14	62.9	10.00	21.10
16	77.6	31.50	16.00
17	78.7	17.50	9.05
19	87.3	27.30	18.40
20	88.8	32.00	20.88
21	93.0	16.00	8.29
22	100.0	28.00	9.84

Table 3-4

Factors for Conversion of Six-Hour Rainfall

Duration to other Duration

Duration Hours	Ratio*
6	1.000
7	1.035
8	1.065
9	1.090
10	1.115
11	1.140
12	1.160
13	1.185
14	1.200
15	1.220
16	1.235
17	1.255
18	1.270
19	1.280
20	1.300
21	1.315
22	1.325
23	1.340
24	1.350
25	1.360
26	1.375
27	1.385
28	1.395
29	1.410
30	1.420
31	1.425
32	1.435
33	1.445
34	1.455
35	1.465
36	1.470

*From the Engineering Handbook, Hydrology, Soil Conservation Service, U.S.D.A.

Note information on durations less then 6 hours may be found in US weather bureau technical paper No. 40.

3.5 Soil Information for Indiana

The soil classification used in the runoff coefficient study was taken from the "The Agronomy Handbook" (25) published in 1961 by Purdue University Agricultural Service. A map taken from this handbook indicating the different soil types is reproduced in Fig. 3-5.

A qualitative description of the permeabilities of the various soil types shown on the map in Fig. 3-5 was given in a report by D. J. Belcher, L. E. Gregg and K. B. Woods. (26) Table 3-5 gives a list of soil types and corresponding permeabilities based on the above report.

Table 3-5 Qualitative Permeabilities of Various Soil Types in Indiana

Soil type as per soil map	Qualitative permeability
A, (H)	very permeable
D, H, O	mostly permeable
(B), C, E, G, M, P	moderately permeable
K, L, N	slowly permeable
B, F, I, J	very slowly permeable

Principal Soil Types of the Regions

A — Maumee, Granby, Newton & Runnymede sandy loams; Plainfield & Tyner sands; mucks; Door Tracey, Fox, Warsaw & Oshtemo loams & sandy loams.

B — Lenawee, Pewamo & Julian silty clay loams; Hoytville silty clay; Rensselaer & Jasper loams & Strole silt loam.

C — Parr & Odell silt loams & loams; Sidell, Raub, Elliott & Flanagan silt loams; Chalmers & Romney silty clay loams.

D — Miami, Crosby, Brookston, Bremen, Galena, Otis, Fox, Fox kame phase & Hillsdale loams & sandy loams; Coloma or Spinks loamy sands.

E — Crosby & Miami silt loams; Brookston & Kokomo silty clay loams.

F — Blount, Morley, Nappanee & St Clair silt loams; Pewamo silty clay loam.

G — Fincastle, Russell & Cope silt loams; Brookston & Kokomo silty clay loams.

H — Genesee, Eel, Huntington, Fox, Ockley, Warsaw, Bartle & Elkinsville silt loams & loams; Westland silty clay loam; Sharkey clay.

I — Cincinnati, Gibson, Vigo, Iva, Wilbur, Stendal & Philo silt loams.

J — Cincinnati, Rossmoyne, Avonburg, Clermont, Jennings, Grayford, Philo, Stendal & Atkins silt loams.

K — Switzerland & Allensville silt loams; Fairmount & Huntington silty clay loams.

L — Muskingum stony loam, Zanesville, Wellston, Tilsit, Elkinsville, Bartle, Otwell & Philo silt loams.

M — Frederick Bewleyville; Bedford, Lawrence, Crider, Pembroke & Huntington silt loams.

N — Otwell, Haubstadt, Dubois, Robinson; Markland, McGary, Henshaw & Parke silt loams; Zipp, Montgomery & Patton silty clay loams.

O — Bloomfield loamy sands; Princeton & Ayrshire sandy loams, loams & silt loams.

P — Alford, Muren, Iva, Hosmer, Adler & Ragsdale silt loams.

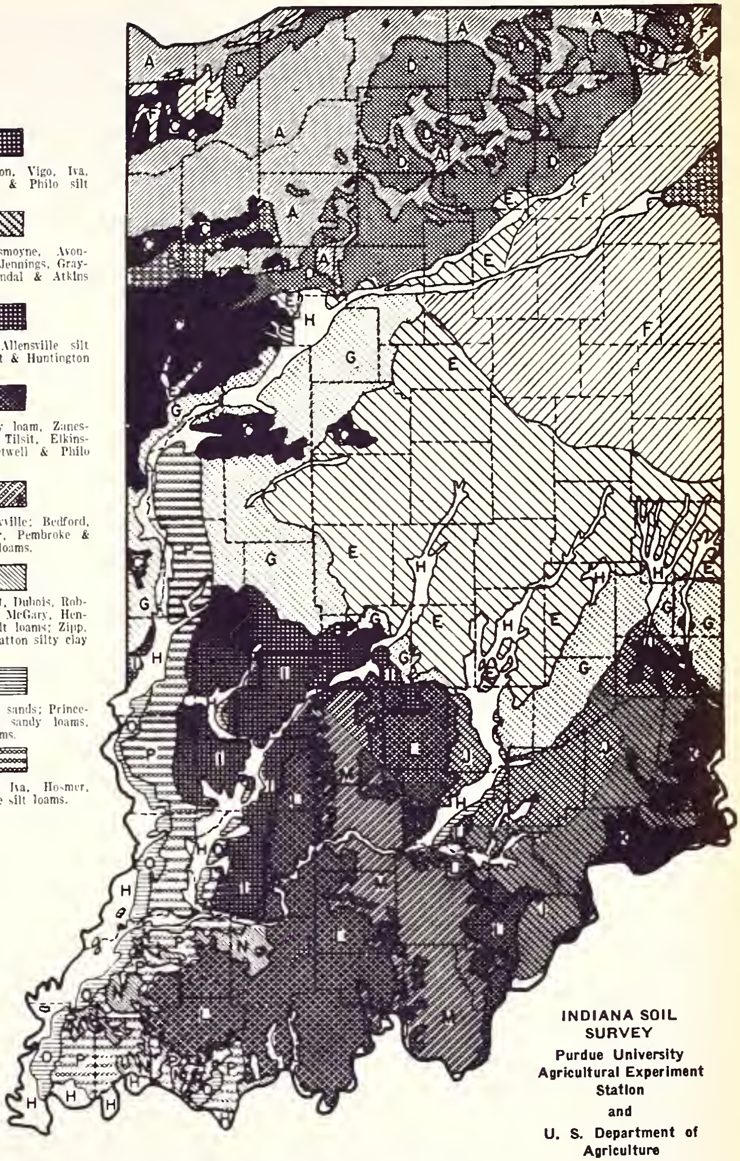


Fig. 3-5 Soil regions of Indiana.

4. DESIGN PEAK DISCHARGE FOR SMALL WATERSHEDS

4.1 Design Peak Discharge for Gaged Watersheds

Data from thirty-two watersheds were used for the frequency analysis of annual peak discharges, as mentioned in Art. 3.2. The results obtained by the method of extreme values analysis are shown in appendix A in the form of plots of annual peak discharge vs. return period on probability paper. The predicted annual instantaneous peak discharges for return periods of 25, 50, 75 and 100 years obtained from the figures of appendix A are listed in Table 4-1.

4.2 The Simple Formula for Peak Discharge from Small Watersheds

The 25-year annual peak discharge Q was found to be related to the watershed area A and the mean slope of main stream S by the formula:

$$Q = 0.000783\ A^{2.63}\ S^{1.54} \qquad (4\text{-}1)$$

in which Q is in cubic feet per second, A is in square miles and S in feet per 10,000 feet. The above relationship was obtained by the method of multiple correlation.

4.3 Working Chart for Peak Discharge Determination by the Simple Formula.

A working chart based on the simple formula 4-1 is given in Fig. 4-1. The 25-year peak discharge can be read directly from the chart knowing the watershed area A and mean slope of the main stream S. An example illustrating the use of these charts is given in Art. 7.1.

4.4 The Extended Formula for Peak Discharge from Small Watersheds

The extended formula for the 25-year annual peak discharge expresses the discharge as a function of five measureable watershed characteristics. The equation was found to be:

$$Q = 0.0718\ A^{0.91414}\ H^{0.80415}\ S^{0.53716}\ D^{0.81865}\ F^{0.43559} \qquad (4\text{-}2)$$

where Q is the 25-year peak discharge, in cfs.

A is the watershed area, in square miles.

H is the mean relief, in feet.

D is the drainage density, in miles per square miles.

F is the watershed shape factor, dimentionless.

S is the main stream slope, in feet per 10,000 feet.

The above formula was also obtained by the method of multiple correlation.

4.5 Working chart for the 25-year peak discharge by the extended formula

A working chart based on formula 4-2 is given in Fig. 4-2. The 25-year peak discharge may be read directly knowing the five watershed characteristics; A, H, D, S, F. An example illustrating the use of the working chart is given in Art. 7.1.

4.6 Peak Discharge for Other Return Periods

In the preceding paragraphs, the peak discharge from small watersheds were obtained for a return period of 25 years. However, it may be desirable to estimate the peak discharge for other return periods so that the design engineer may have a greater freedom of choice. The relationship between the peak discharge for other frequency and the 25-year peak discharge can be obtained from Gumbel's extreme value theory. Fig. 4-3 gives the relationship between the 25-year peak discharge and the values of peak discharge for frequencies of 10, 50, 75 and 100 years.

4.7 An Estimate of the Accuracy of Peak Discharge Determination.

The most accurate method of estimating the peak discharge for a given return period is by means of a frequency analysis of the flow records, if such records are available for the site under consideration. This may be called the direct method. If the duration of the flow records is sufficiently long (say 15 years

or more), the frequency analysis yields a good estimate of the peak discharge. This method is, of course, possible only for gaged watersheds. For ungaged watersheds indirect methods have to be used. The estimate of the peak discharge by correlation to watershed characteristics is always less accurate than the direct method, provided data for the latter exist.

The estimate of peak discharge by means of regression formulas based on a correlation analysis is subject to two kinds of errors. The first error is that resulting from the use of records of short duration in the frequency analysis. The source of the second error is the choice of the correlation variables and the size of the sample (number of watersheds) on which the correlation is based.

An estimate of the error due to the selection of variables can be obtained by comparing the original values of peak discharge obtained by means of the frequency analysis and the corresponding values computed by the simple and extended formulas. From this comparison shown in Tables 4-2 and 4-3, the mean deviations were found to be about 4,900 cfs for the simple formula and about 2,400 cfs for the extended formula. Figures 4-4 and 4-5 show plots of the estimates of the 25-year peak discharge by means of the simple and of the extended formula respectively, versus the 25-year peak discharge obtained from the frequency analysis. The reduction of the error of estimate of the peak discharge by means of the extended formula may be seen by comparisonn of the two figures.

These errors of estimate should be kept in mind by the designing engineer. The methods proposed should be used as an aid to engineering judgement rather than a replacement of engineering judgement.

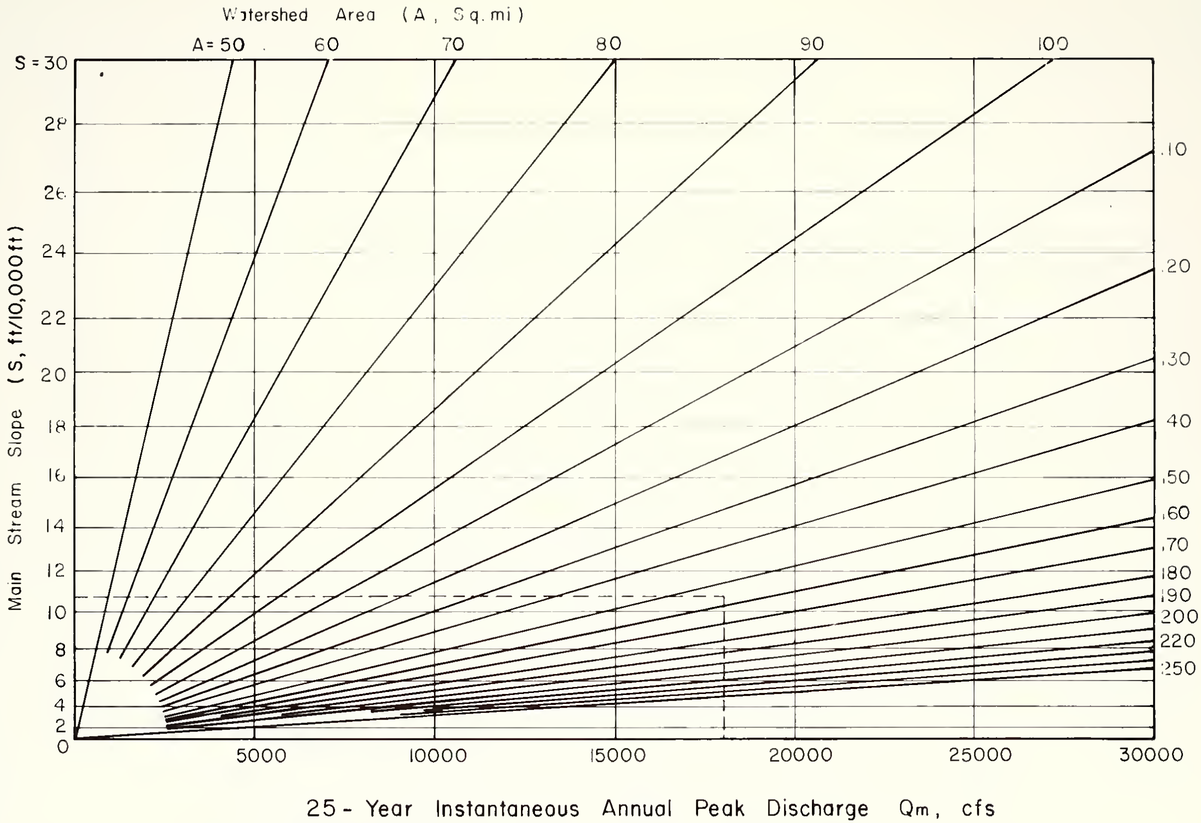


FIG. 4-1 WORKING CHART FOR PEAK DISCHARGE DETERMINATION BY SIMPLE FORMULA (EQ. 4-1).

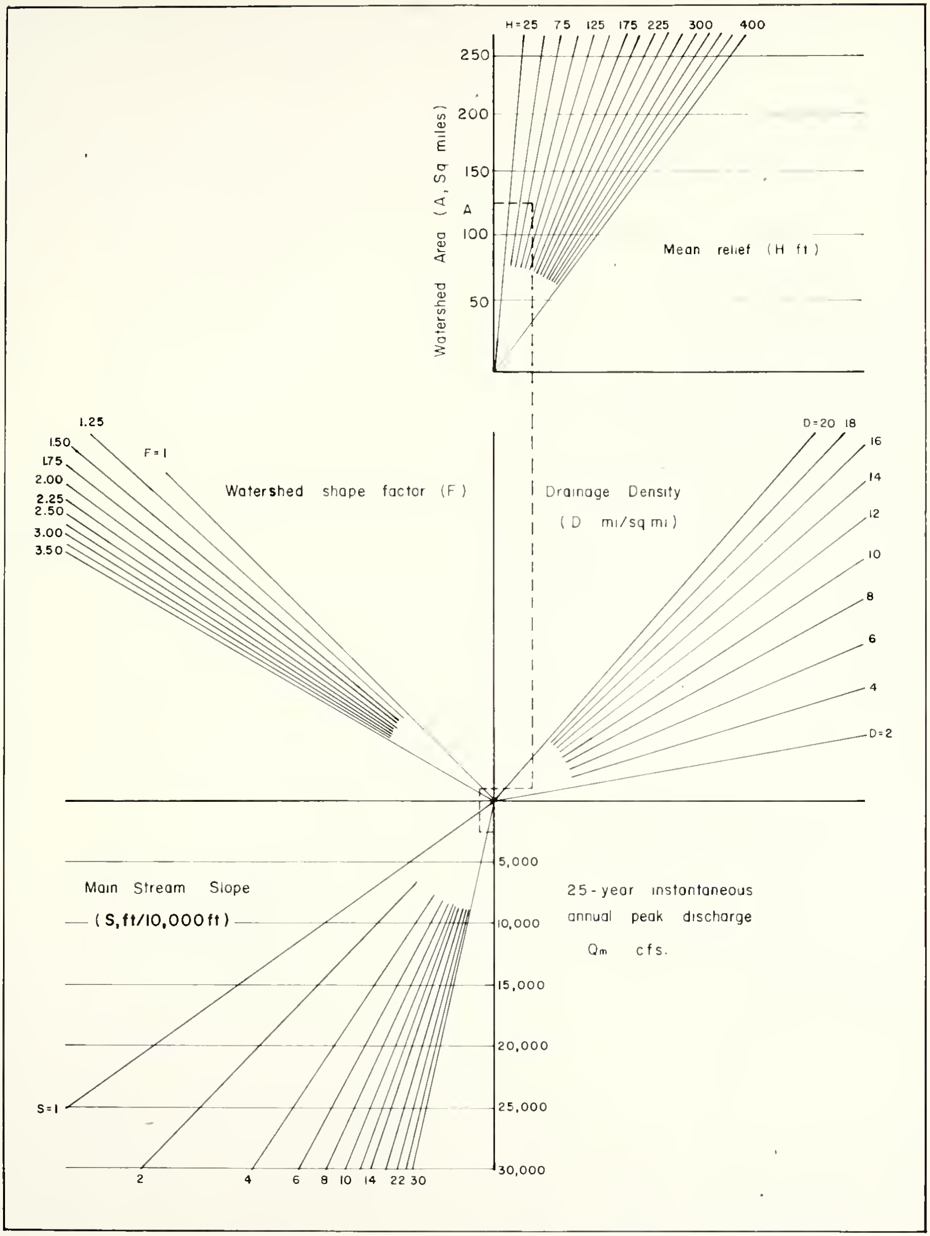


FIG. 4-2 WORKING CHART FOR PEAK DISCHARGE DETERMINATION BY EXTENDED FORMULA

(EQU 4-2)

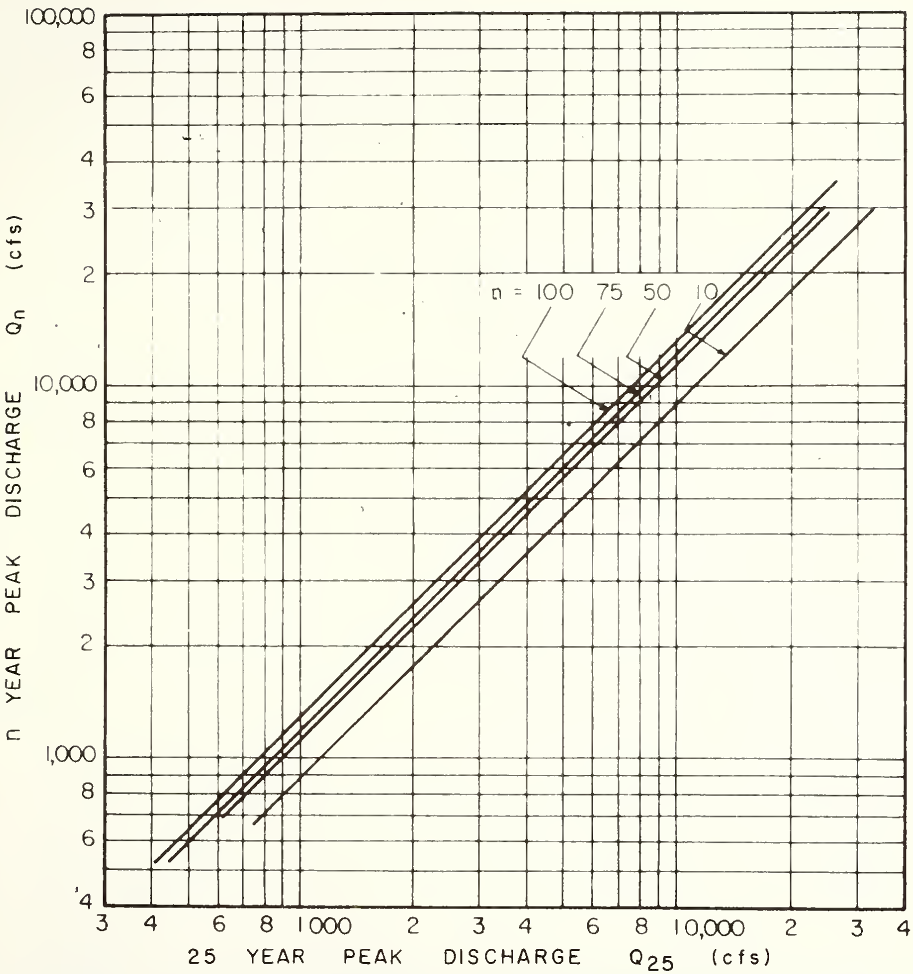


FIG. 4–3 RELATIONSHIP BETWEEN THE n-YEAR AND THE 25-YEAR PEAK DISCHARGE

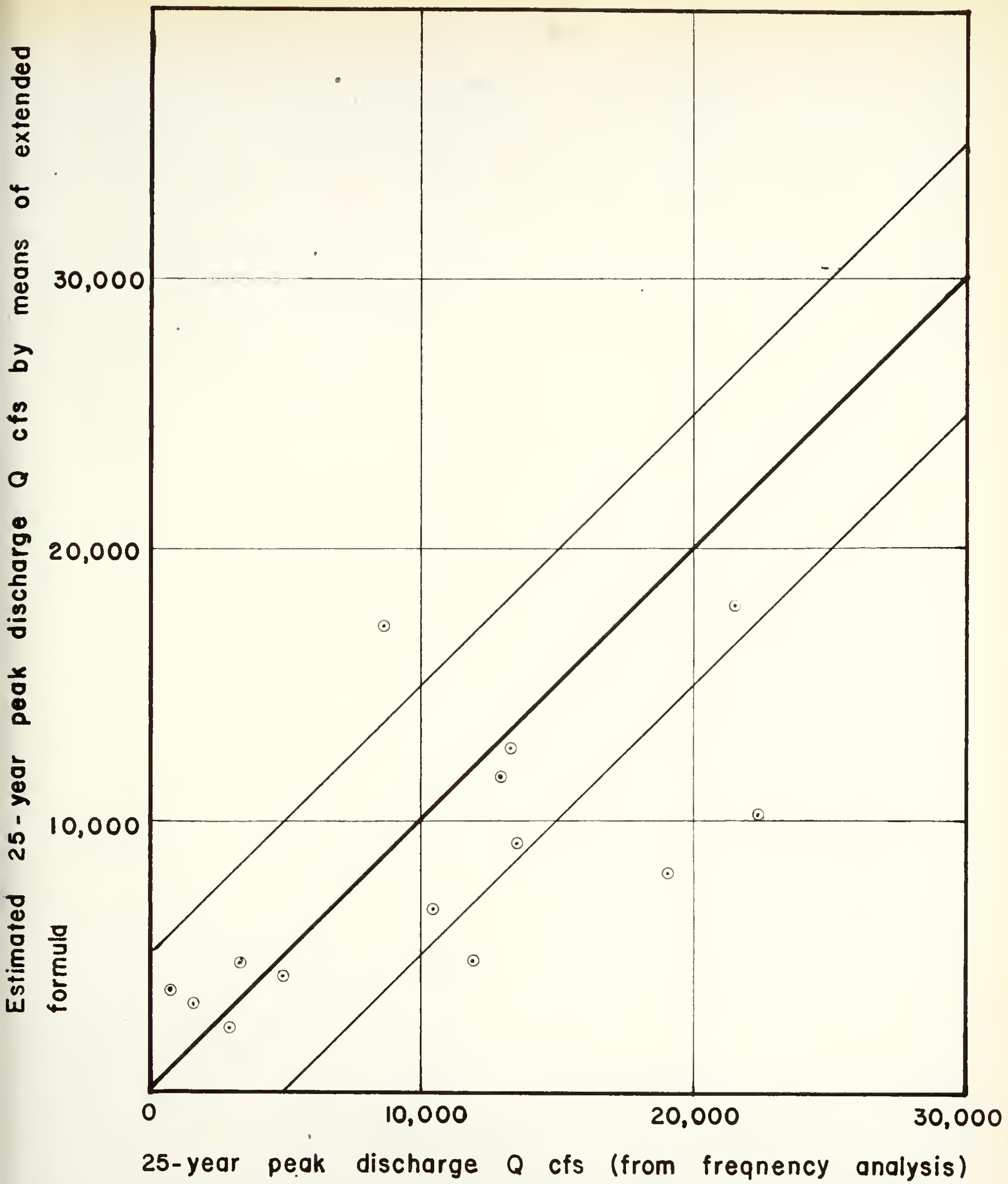


FIG. 4-4 COMPARISON OF 25 YEAR PEAK DISCHARGE ESTIMATED BY SIMPLE FORMULA WITH ORIGINAL VALUES

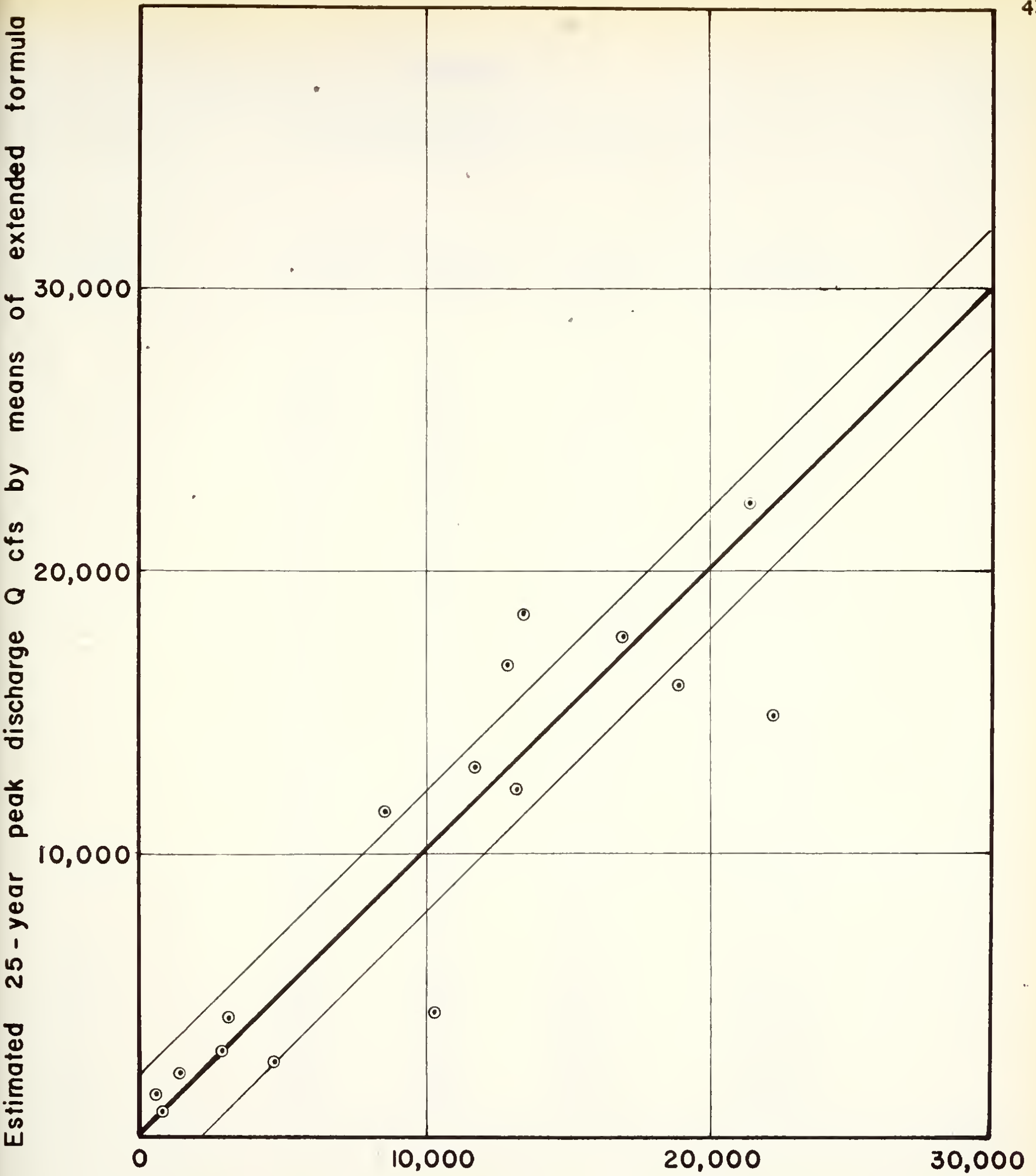


FIG. 4-5 COMPARISON OF 25 YEAR PEAK DISCHARGE ESTIMATED BY EXTENDED FORMULA WITH ORIGINAL VALUES

Table 4-1

The Predicted Annual Instantaneous Peak Discharge
from Flood Frequency Analysis

Watershed Number	Predicted Annual Instantaneous Peak Discharge			
	25-years cfs.	50-years cfs.	75-years cfs.	100-years cfs.
6	355	925	960	1010
7	465	520	[illegible]	570
11	4160	4800	5[illegible]00	5500
12	2100	2350	2490	2600
14	3300	3800	4000	4300
15	3300	3750	3940	4200
17	2950	3420	3600	3860
18	2440	2770	2900	3080
19	20500	24000	25400	27000
20	12900	15400	16400	17300
21	1630	1800	1880	1980
22	11800	13200	14700	15900
23	760	860	900	960
24	10500	12400	13400	14000
25	880	1000	1040	1110
26	19000	22000	23700	25200
27	1270	1370	1410	1470
28	19800	22800	24800	26000
29	4300	5700	5700	6200
30	22300	25700	27800	29400
31	14000	16[illegible]00	17[illegible]00	18800
32	6800	7700	8[illegible]00	8900
33	1020	1130	1170	1230
34	21500	24[illegible]00	25[illegible]00	26800
35	7400	8[illegible]00	9[illegible]00	9700
36	7700	8[illegible]00	9[illegible]00	10000
37	17000	19[illegible]00	21000	22300
38	10800	12[illegible]00	13[illegible]00	13600
39	13300	15[illegible]00	15900	17000
40	8700	9[illegible]00	10[illegible]00	11000
41	28600	33000	35[illegible]00	37300
42	13500	15[illegible]00	16[illegible]00	17[illegible]00

TABLE 4-2

Comparison of the Estimates of Peak Discharge by Means of the Frequency Analysis and by the Simple Formula (Eq 4-1)

Watershed No	Peak Discharge Frequency Analysis	Peak Discharge Eq 4-1	Deviation
14	3300	4706	1406
17	2950	2296	654
20	12900	11463	1437
21	1630	3110	1480
22	11800	4903	6897
23	760	3702	2942
24	10500	6688	3812
25	880	929	49
26	19000	7992	11008
29	4800	4160	640
30	22300	10060	12240
34	21500	17907	3593
37	17000	36488	19488
39	13300	12611	639
40	8700	17088	8388
42	13500	9126	4374

Mean Deviation 4940 cfs

TABLE 4-3

Comparison of the Estimates of Peak Discharge by Means of the Frequency Analysis and by the Extended Formula (Eq 4-2)

Watershed No	Peak Discharge Frequency Analysis	Peak Discharge Eq 4-1	Deviation
14	3300	4112	812
17	2950	3091	141
20	12900	16588	3688
21	1630	2124	494
22	11800	13163	1363
23	760	1488	728
24	10500	4351	6149
25	880	834	46
26	19000	16130	2870
29	4800	2514	2286
30	22300	14929	7371
34	21500	22382	882
37	17000	17619	619
39	13300	12467	833
40	8700	11367	2667
42	13500	18384	4884

Mean Deviation 2240 cfs

5. DESIGN HYDROGRAPHS FOR SMALL WATERSHEDS

5.1 The Two Parameters Equation for the Short Duration Unit Hydrograph

Short duration hydrographs for small watersheds have a characteristic shape showing a quick rise to peak and a relatively slower recession. An equation suitable for the mathematical description of such curves is that proposed by some investigators (16,17) for the instantaneous unit hydrograph.

$$Q = \frac{640\,AR}{K\,\Gamma(n)} \left(\frac{t}{K}\right)^{n-1} e^{-t/K} \qquad (5\text{-}1)$$

In this equation Q is the discharge in cfs, t is the time in hours after the beginning of direct surface runoff, A is the area of the watershed in square miles, R is the total runoff in inches and the quantities K and n are the parameters of the equation. K has the dimensions of time and is expressed in units of hours, and n is a dimensionless number. The quantity $\Gamma(n)$ is the gamma function, the value of which depends on the value of n. For integer values of n, the value of the gamma function is given by

$$\Gamma(n) = (n-1)\cdot(n-2)\cdot(n-3)\cdot \ldots \cdot 1$$

or

$$\Gamma(n) = (n-1)! \qquad (5\text{-}2)$$

Values of the gamma function for non-integer values of n are given in Table 5-1.

By differentiating Equation 5-1 with respect to t and making dQ/dt = 0, it can be shown that the time to peak in equation 5-1 is given by

$$t_p = (n-1)\,K \qquad (5\text{-}3)$$

Using the time to peak as a basis for dimensionless ratios, equation (5-1) may be rewritten as

$$\frac{Q\,t_p}{640\,AR} = \frac{(n-1)^n}{\Gamma(n)} \left[\left(\frac{t}{t_p}\right) e^{-t/t_p}\right]^{n-1} \qquad (5\text{-}4)$$

showing that time to peak can be used instead of K as one of the parameters of the equation.

Table 5-1

Values of the Gamma Function

n	(n)	n	$\Gamma(n)$
1.0	1.000	3.0	2.000
1.1	0.951	3.25	2.549
1.2	0.918	3.50	3.323
1.3	0.897	3.75	3.423
1.4	0.887	4.0	6.00
1.5	0.886	4.5	11.63
1.6	0.894	5.0	24.00
1.7	0.909	5.5	52.33
1.8	0.931	6.0	120.00
1.9	0.961	6.5	287.8
2.0	1.000	7.0	720.0
2.2	1.102	7.5	1870.7
2.4	1.242	8.0	5040.0
2.6	1.430	9.0	40320
2.8	1.676		

The value of the second parameter (n) can be estimated by comparing the recession curves of the actual hydrograph and that given by Equation 5-4. Plotting the recession curve of the actual hydrograph on semi-logarithmic paper, with discharge plotted on the loganthmic scale, it is possible to fit a straight line to the part of the curve immediately following the crest section of the hydrograph. The dimensionless recession constant (K_1/t_p) is then estimated from this line by the equation

$$\frac{K_1}{t_p} = \frac{t_1 - t_o}{2.3\, t_p \log (Q_o/Q_1)} \qquad (5\text{-}5)$$

In this equation, t_p is the time to peak of the hydrograph, Q_o and Q_1 are two values of discharge and t_1 and t_o are the corresponding two values of time, which are read from any two points on the straight line in the semi-logarithmic plot.

The above procedure was used also to determine the recession constants of the dimensionless hydrographs obtained from equation 5-4 as the value of n was varied. The values of the demensionless recession constants obtained for various values of n were plotted on a diagram (Fig. 5-1) showing the relationship between the two quantities. Such a diagram can be used for estimating the value of the parameter n when the quantity K_1/t_p is known.

An alternative method for estimating the value of n could be the comparison of the actual hydrographs, plotted dimensionlessly as (Q/Q_p) versus (t/t_p), with a set of similar curves obtained from Equation 5-4 by assuming a set of various values of the parameter n. A set of such curves is given in Fig. 5-2 and a listing of the values of the variables from which the diagram has been plotted is given in Table 5-2.

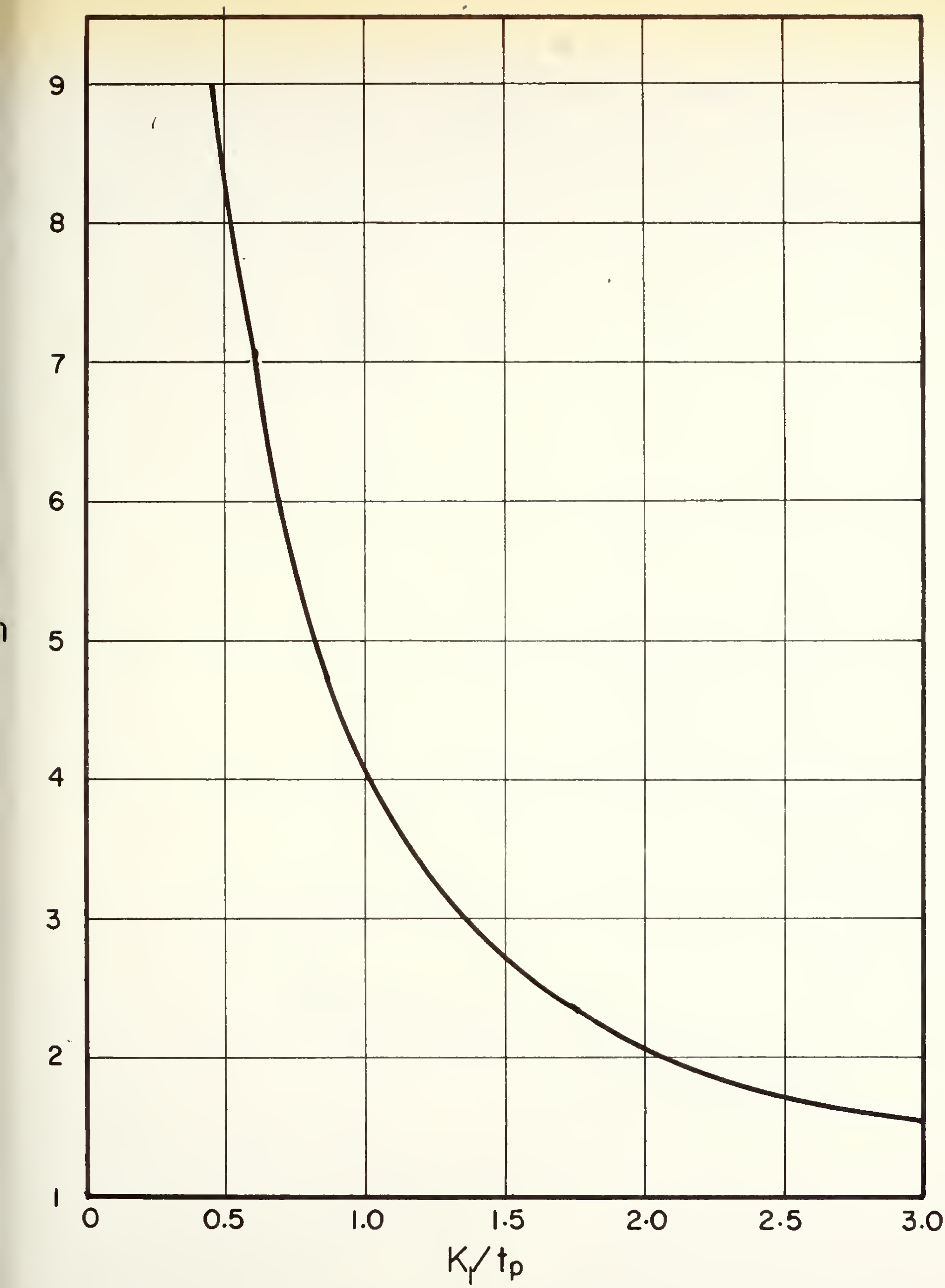


FIG 5-1 RELATIONSHIP BETWEEN DIMENSIONLESS RECESSION CONSTANT AND HYDROGRAPH PARAMETER

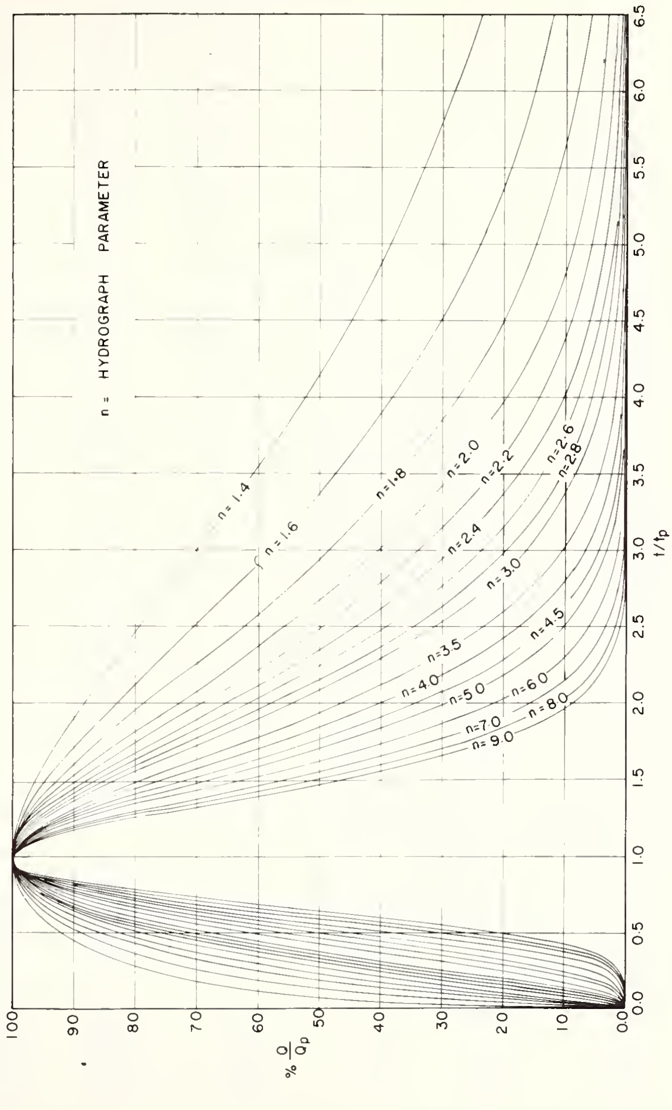


FIG 5-2 DIMENSIONLESS INSTANTANEOUS HYDROGRAPH

Table 5-2 The Dimensionless Instantaneous Hydrograph

Q/Q_p (%)

t/t_p	n = 1.4	1.6	1.8	2.0	2.2	2.4
0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.1	57.1	43.1	32.6	24.6	18.6	14.0
0.2	72.3	61.5	52.3	44.5	37.9	32.2
0.3	81.7	73.9	66.8	60.4	54.6	49.4
0.4	88.1	82.7	77.6	72.9	68.4	64.2
0.5	92.6	89.1	85.7	82.4	79.3	76.3
0.6	95.7	93.6	91.5	89.5	87.6	85.6
0.7	97.8	96.7	95.6	94.5	93.4	92.4
0.8	99.1	98.6	98.2	97.7	97.3	96.8
0.9	99.8	99.7	99.6	99.5	99.4	99.2
1.0	100.0	100.0	100.0	100.0	100.0	100.0
1.1	99.8	99.7	99.6	99.5	99.4	99.3
1.2	99.3	98.9	98.6	98.2	97.9	97.6
1.3	98.5	97.8	97.0	96.3	95.6	94.9
1.4	97.5	96.3	95.0	93.8	92.7	91.5
1.5	96.3	94.5	92.7	91.0	89.3	87.6
1.6	94.9	92.5	90.1	87.8	85.6	83.4
1.7	93.4	90.3	87.3	84.4	81.6	78.9
1.8	91.9	83.0	84.4	80.9	77.5	74.3
1.9	90.2	85.6	81.3	77.2	73.4	69.7
2.0	88.4	83.2	78.2	73.5	69.2	65.1
2.2	84.8	73.1	72.0	66.3	61.0	56.2
2.4	81.1	73.0	65.7	59.2	53.3	48.0
2.6	77.3	67.9	59.7	52.5	46.1	40.6
2.8	73.5	63.0	54.0	46.3	39.7	34.0
3.0	69.7	53.2	48.6	40.6	33.9	28.3
3.5	60.7	47.3	36.9	28.7	22.3	17.4
4.0	52.4	33.0	27.5	19.9	14.4	10.4
4.5	45.0	30.2	20.3	13.6	9.1	6.1
5.0	38.4	23.8	14.8	9.2	5.7	3.5
5.5	32.7	13.7	10.7	6.1	3.5	2.0
6.0	27.7	14.6	7.7	4.0	2.1	1.1
6.5	23.4	11.3	5.5	2.7	1.3	0.6
7.0	19.8	8.8	3.9	1.7	0.8	0.3
7.5	16.6	6.8	2.8	1.1	0.5	0.2

Table 5-2 (Continued)

t/t_p	n=2.6	2.8	3.0	3.5	4.0	4.5
0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0	10.6	8.0	6.0	3.0	1.5	0.7
0.2	27.4	23.3	19.8	13.2	8.8	5.9
0.3	44.6	40.4	36.5	28.4	22.0	17.1
0.4	60.3	56.6	53.1	45.4	38.7	33.0
0.5	73.4	70.6	68.0	61.7	56.0	50.9
0.6	83.8	81.9	80.1	75.8	71.7	67.8
0.7	91.3	90.3	89.3	86.8	84.4	82.0
0.8	96.4	95.9	95.5	94.4	93.3	92.2
0.9	99.2	99.0	98.[illegible]	98.7	98.4	98.1
1.0	100.0	100.0	100.0	100.0	100.0	100.0
1.1	99.2	99.2	99.[illegible]	98.8	98.6	98.4
1.2	97.2	96.9	96.[illegible]	95.7	94.8	94.0
1.3	94.2	93.4	92.3	91.0	89.3	87.7
1.4	90.3	89.2	88.[illegible]	85.3	82.6	80.1
1.5	86.0	84.[illegible]	[illegible]	79.0	75.3	71.8
1.6	81.2	79.[illegible]	77.[illegible]	72.2	67.7	63.4
1.7	76.3	73.[illegible]	71.[illegible]	65.5	60.2	55.3
1.8	71.2	68.2	65.[illegible]	58.8	52.9	47.6
1.9	66.2	62.8	59.[illegible]	52.4	46.1	40.5
2.0	61.2	57.6	54.[illegible]	46.4	39.8	34.2
2.2	51.8	47.[illegible]	43.9	3[illegible].7	29.1	23.7
2.4	43.2	38.[illegible]	35.0	2[illegible].9	20.7	16.0
2.6	35.7	31.4	27.0	20.0	14.9	10.5
2.8	29.2	25.0	21.[illegible]	14.6	9.9	6.8
3.0	23.6	19.7	16.5	10.5	6.7	4.3
3.5	13.6	10.[illegible]	[illegible]	4.4	2.4	1.3
4.0	7.6	5.[illegible]	4.[illegible]	1.8	0.8	0.4
4.5	4.1	2.[illegible]	1.[illegible]	0.7	0.2	0.1
5.0	2.2	1.[illegible]	0.8	0.2	0.1	0.0
5.5	1.1	0.6	0.4	0.1	0.0	
6.0	0.6	0.3	0.2	0.0		
6.5	0.3	0.2	0.1			
7.0	0.2	0.1	0.0			
7.5	0.1	0.0				

Table 5-2 (Continued)

t/t_p	n = 5.0	6.0	7.0	8.0	9.0
0.0	0.0	0.0	0.0	0.0	0.0
0.1	0.4	0.1	0.0	0.0	0.0
0.2	3.9	1.8	0.8	0.4	0.2
0.3	13.3	8.0	4.9	2.9	1.8
0.4	28.2	20.6	15.0	10.9	8.0
0.5	46.2	38.1	31.4	25.9	21.3
0.6	64.2	57.5	51.4	46.0	41.2
0.7	79.7	75.3	71.2	67.2	63.6
0.8	91.2	89.1	87.0	85.0	83.1
0.9	97.9	97.4	96.8	96.3	95.8
1.0	100.0	100.0	100.0	100.0	100.0
1.1	98.1	97.7	97.2	96.8	96.3
1.2	93.2	91.5	89.9	88.4	86.8
1.3	86.0	82.8	79.8	76.8	74.0
1.4	77.6	72.8	68.3	64.1	60.2
1.5	68.5	62.3	56.7	51.6	46.9
1.6	59.4	52.2	45.8	40.2	35.4
1.7	50.8	42.9	36.2	30.6	25.8
1.8	42.8	34.6	28.0	22.6	18.3
1.9	35.6	27.5	21.2	16.4	12.7
2.0	29.3	21.6	15.9	11.7	8.6
2.2	19.3	12.8	8.5	5.6	3.7
2.4	12.3	7.3	4.3	2.5	1.5
2.6	7.6	4.0	2.1	1.1	0.6
2.8	4.6	2.1	1.0	0.4	0.2
3.0	2.7	1.1	0.4	0.2	0.1
3.5	0.7	0.2	0.1	0.0	0.0
4.0	0.2	0.0	0.0		
4.5	0.0				
5.0					
5.5					

5.2 Estimation of the Time to Peak and of the Recession Constant from Physical Characteristics

Records of total hydrographs for 5 to 6 storms on each of the 17 watersheds listed in Table 3-1, Section 3.3, were obtained; the direct surface runoff hydrographs were derived from the total hydrographs and reduced to a dimensionless form (Q/Q_p versus t/t_p) as described in Section 2.4. Comparing the dimensionless hydrographs obtained from various storms for any one watershed, it was found that the values of the time to peak were approximately equal and that the dimensionless curves plotted for the various hydrographs had approximately the same characteristic shape. Table 5-3 lists the values of the time to peak t_p, and the value of the recession constant K_1, and the corresponding value of the parameter n for each of the watersheds studied.

A multiple correlation analysis was carried out to determine the relationship between each of the quantities t_p and K_1, and the physical features of the watershed. The features considered were the area A, the length of main stream L, and the slope of the main stream S. The values of these characteristics are given in Table 3-3.

The equations obtained from the multiple correlation analysis were

$$t_p = 51.4\, A^{1.08}\, L^{-1.23}\, S^{-0.67} \qquad (5\text{-}6)$$

$$K_1 = 783\, A^{0.94}\, L^{-1.48}\, S^{-1.47} \qquad (5\text{-}7)$$

The agreement between the measured quantities of t_p and K_1 and the theoretical line given by Equations 5-6 and 5-7 is indicated in Figs. 5-3 and 5-4 respectively. The mean deviation between the measured values of t_p and those computed by Equation 5-6 was 3.6 hours, and the meandeviation between measured values of K_1 and those computed by Equation 5-7 was 2.5 hours.

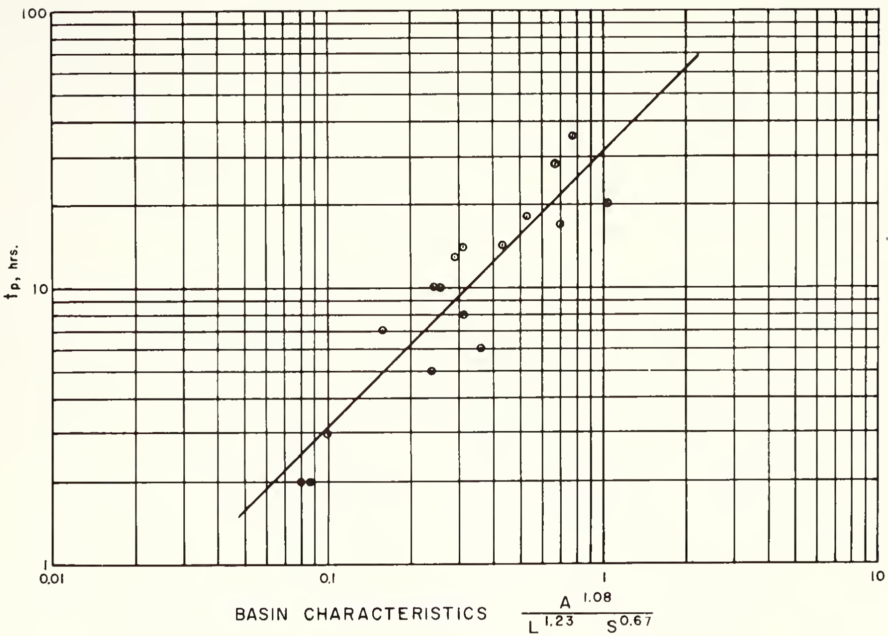


FIG. 5-3 RELATIONSHIP BETWEEN tp AND WATERSHED CHARACTERISTICS A, L, & S

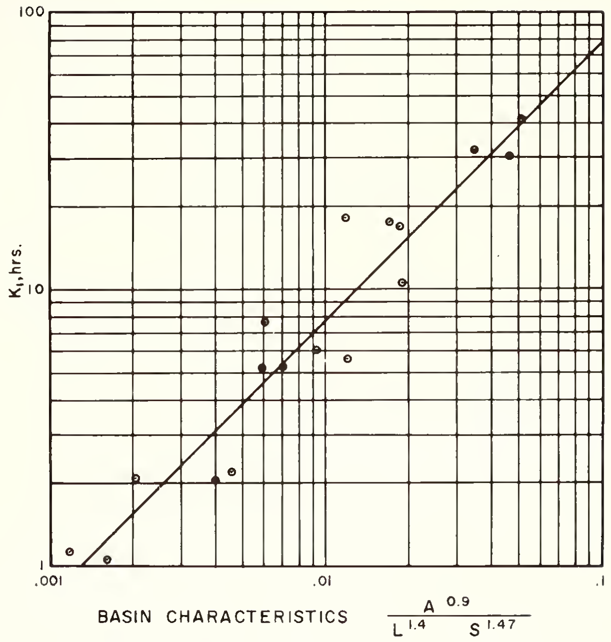


FIG 5- 4 RELATIONSHIP BETWEEN K_1 AND WATERSHED CHARACTERISTICS A, L, & S

Table 5-3

HYDROGRAPH PARAMETERS

Watershed number	Time to peak t_p (hr)	Recession constant K_1 (hr)	Hydrograph Parameter n
1	2	1.13	7
2	2	1.04	7
3	3	2.06	6
4	7	2.04	10
5	10	6.00	7
8	6	5.60	5
9	14	18.00	3.5
10	14	17.60	3
12	17	30.00	2.5
13	13	5.20	10
14	28	17.00	7
16	10	7.60	5.5
17	35	32.00	5
19	8	5.30	6
20	5	2.20	8
21	20	40.70	1.9
22	18	10.65	7

5.3 Working Charts for Determination of Hydrograph Parameters

If the values of the area of a watershed, the length of the main stream and the slope of the main stream are available or can be measured from a topographic map, it is possible to derive the unit hydrograph of short duration from the given watershed. The procedure would be to estimate the values of t_p and K_1 from Equations 5-6 and 5-7, to determine the corresponding value of n from Fig. 5-1 and then to plot the dimensionless unit hydrograph from Fig. 5-2 or from data in Table 5-2. Finally the actual unit hydrograph, in which the discharge is expressed in cfs and the time in hours, can be plotted using the known value of t_p and the value of Q_p computed by the following relationship, obtained from equations 5-3 and 5-4.

$$\frac{Q_p t_p}{640\ AR} = \frac{(n-1)^{n-1}}{\Gamma(n)} e^{-(n-1)} \qquad (5\text{-}8)$$

in which R is taken to be 1 inch (R = 1). Values of the quantity ($Q_p t_p/640$ AR) as a function of the hydrograph parameter n, computed by equation 5-8, are given in Table 5-4.

The relationship between the dimensionless peak discharge and the hydrograph parameter is given also in Fig. 5-5.

As an alternative to the solution of Equations 5-6 and 5-7, diagrams have been prepared from which the values of t_p and K_1 can be read for given values of A, L and S. These diagrams are given in Fig. 5-6 and 5-7.

5.4 Derivation of Unit Hydrographs of Other Durations

The equation used for the description of the unit hydrograph in this report is one originally proposed for instantaneous unit hydrographs. It was taken to apply also for unit hydrographs of definite but short durations, of the order of $0.1t_p$. If it is required to produce a unit hydrograph of longer durations, it is possible to use a graphical or a numerical method for the production of the required unit hydrograph. It is assumed in these methods that the duration of

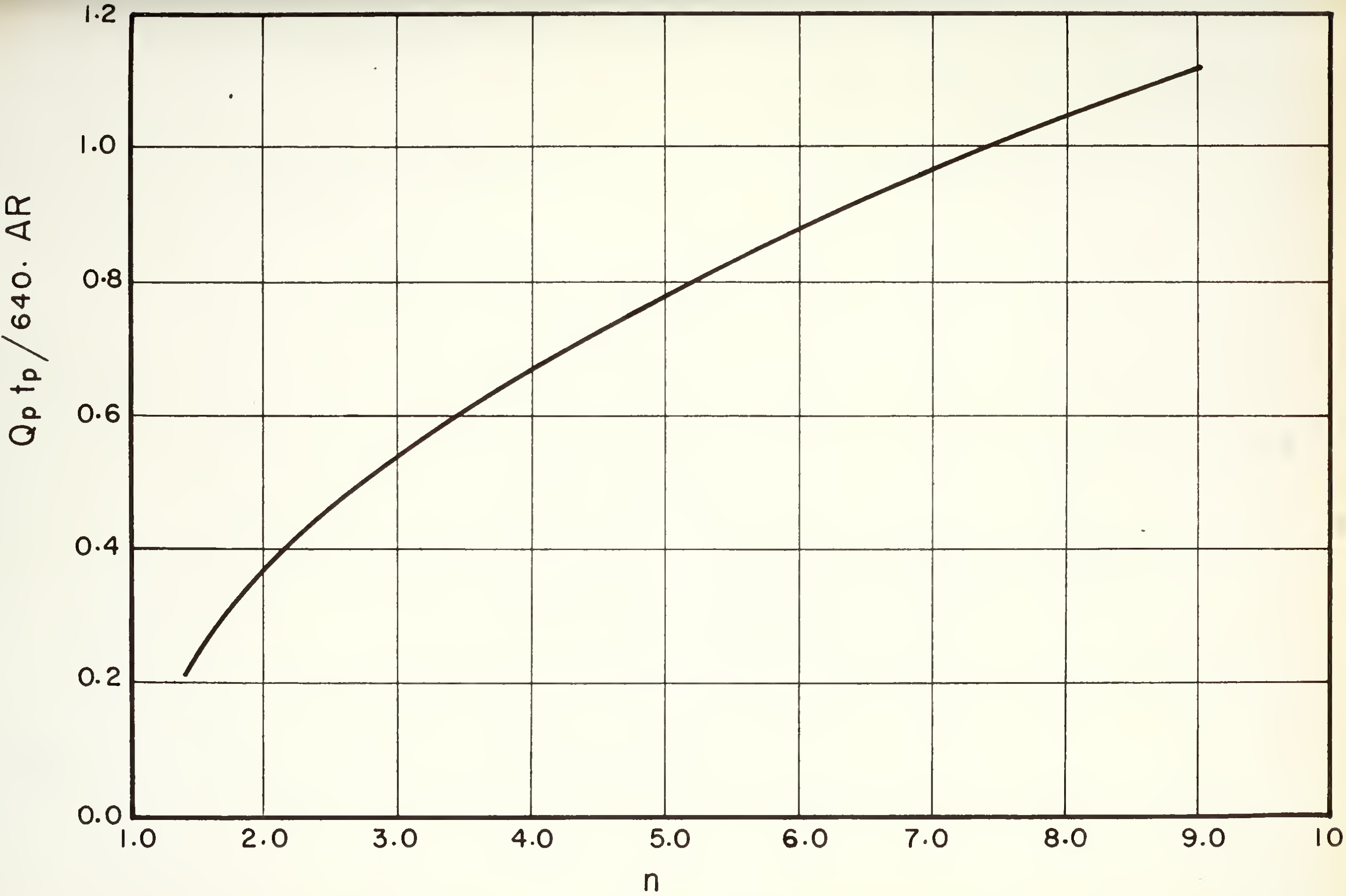


FIG 5—5 RELATIONSHIP BETWEEN DIMENSIONLESS PEAK DISCHARGE AND HYDROGRAPH PARAMETER n

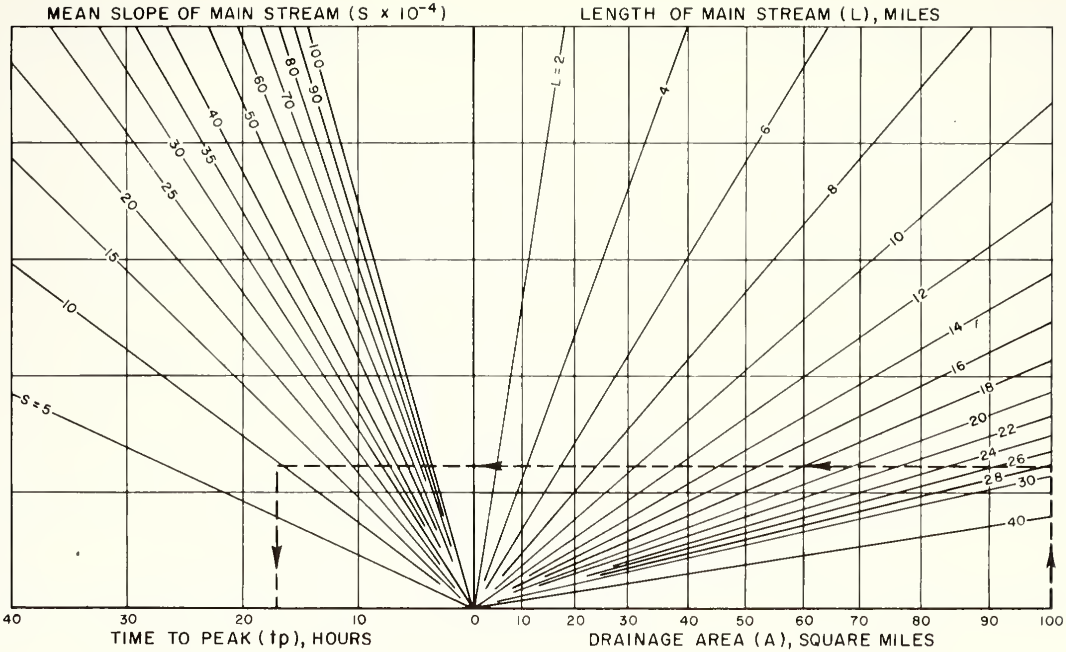


FIG. 5-6 WORKING CHART FOR REGRESSION FORMULA FOR tp

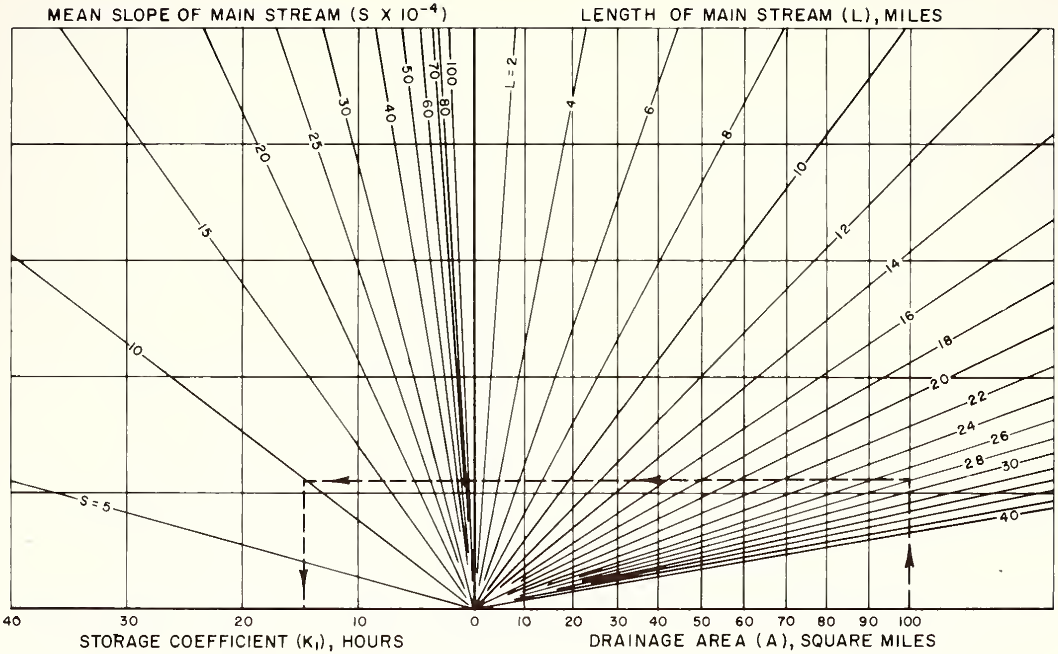


FIG 5-7 WORKING CHART FOR REGRESSION FORMULA FOR K_1

Table 5-4

Values of the Dimensionless Peak Discharge for Various Values of n

n	$\frac{Q_p t_p}{640\ AR}$	n	$\frac{Q_p t_p}{640\ AR}$
1.4	0.210	3.75	0.642
1.6	0.271	4.0	0.672
1.8	0.323	4.5	0.729
2.0	0.368	5.0	0.781
2.2	0.408	5.5	0.831
2.4	0.445	6.0	0.877
2.6	0.479	6.5	0.922
2.8	0.511	7.0	0.964
3.0	0.541	7.5	1.004
3.25	0.577	8.0	1.043
3.50	0.610	9.0	1.117

the resulting unit hydrograph is an exact multiple of the duration of the original unit hydrograph of short duration.

In the graphical method (Fig. 5-8), a number of unit hydrographs are drawn vertically below each other in an offset position. The number of hydrographs drawn is equal to the ratio of the duration of the resulting hydrograph to the duration of the original hydrograph and the amount of horizontal offset of each hydrograph with respect to the one above it is equal to the duration of these unit hydrographs. The ordinates falling on any vertical line are then added for all the offset hydrographs to give the ordinate of the summation curve. Fianlly the ordinates of the summation curve are divided by the number of unit hydrographs involved in the summation to give the required unit hydrograph of the required duration.

In the numerical procedure, the ordinates of the short duration unit hydrograph, corresponding to times T_1, $2T_1$, $3T_1$, . . .,(where T_1 is the duration of the unit hydrograph) are denoted by U_1, U_2, U_3, etc.; the ordinates of the unit hydrograph of longer duration at the same times are denoted by q_1, q_2, q_3, etc. If the duration of the longer unit hydrograph is $T = NT_1$ where N is some integer number, then the relationship between the nth ordinates of the unit hydrograph of longer duration and the ordinates of the short duration unit hydrograph is given by

$$q_n = \frac{1}{N} \sum_{i=1}^{k} U_{n-i+1} \qquad (5-9)$$

where i is the variable of the summation; and k is taken as either $k = n$ or $k = N$ whichever is the smaller of the two numbers

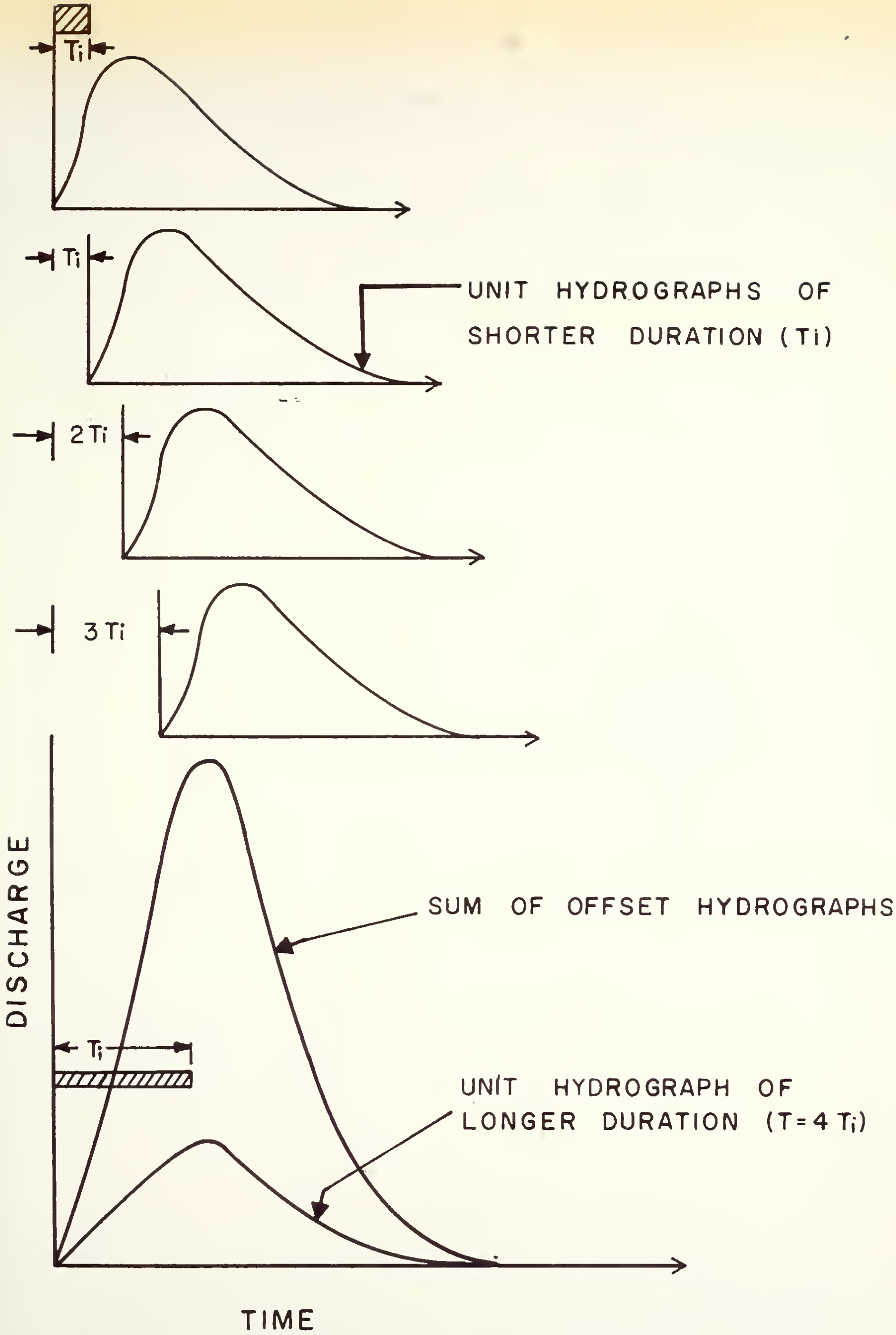


FIG 5-8 DERIVATION OF UNIT HYDROGRAPHS OF LARGER DURATION

5.5 Design Hydrographs from Design Rainfall Hyetographs

The analysis of rainfall records leads to values of the depths of rainfall that can be expected with a given frequency for various durations. From this information it is then possible to construct a design hyetograph of total rainfall giving the depths of rainfall at successive time intervals during a hypothetical storm of the given frequency. Subtracting from this hyetograph the estimated infiltration losses leads to a design hyetograph of rainfall excess.

The derivation of the design hydrograph corresponding to the design hyetograph of rainfall excess is carried out by a summation process based on the assumption of linear relationship between rainfall and runoff. The first step is to derive a unit hydrograph of duration T equal to the time interval used in the construction of the hyetograph blocks (Fig. 2-4). Denoting the depth of rainfall represented by each block in the hyetograph by P_1, P_2, P_3, . . . P_N, where N is the number of blocks in the hyetograph, also denoting the ordinates of the unit hydrograph at times T_1, T_2, T_3, etc. by q_1, q_2, q_3, etc., and the ordinates of the design hydrograph at the same times by Q_1, Q_2, Q_3 etc. the relationship between the nth ordinate of the design hydrograph Q_n and the ordinate of the unit hydrograph is given by:

$$Q_n = \sum_{i=1}^{k} P_{(i)} \, q_{(n-i+1)} \qquad (5\text{-}10)$$

where i is the variable of the summation and k is taken as either k = n or k = N whichever is the smaller of the two quantities.

6. THE RELATIONSHIP BETWEEN RAINFALL AND RUNOFF

6.1 Factors Affecting the Amount of Runoff

There is no definite relationship available for calculating the amount of direct surface runoff resulting from a given rainfall, as the factors affecting the total volume of runoff are numerous and difficult to evaluate. In the process of conversion of rainfall to runoff, infiltration into the ground appears to be the most important single factor affecting the volume of runoff produced by a given rainfall. Some of the factors affecting the infiltration rate are:

A - Climatological conditions:

Rainfall intensity, duration and distribution; initial moisture condition; ground water elevation; and presence of snow or ice cover.

B - Watershed conditions:

Soil types and permeability; ground cover and land use; and physical features of the watershed.

Some of the other factors affecting the volume of runoff are the depression storage, reservoir storage and interception loss, and to a lesser extent also evapotranspiration.

6.2 Definition of Runoff Coefficient

The runoff coefficient r used in this study was defined as the ratio of total volume of runoff R to the volume of rainfall P_x occuring after the beginning of runoff.

$$r = \frac{P_x}{R}$$

where both R and P_x are expressed in inches.

The runoff coefficient for any storm can be obtained from the analysis of runoff hydrograph and the rainfall hyetograph, as shown in fig. 6-1.

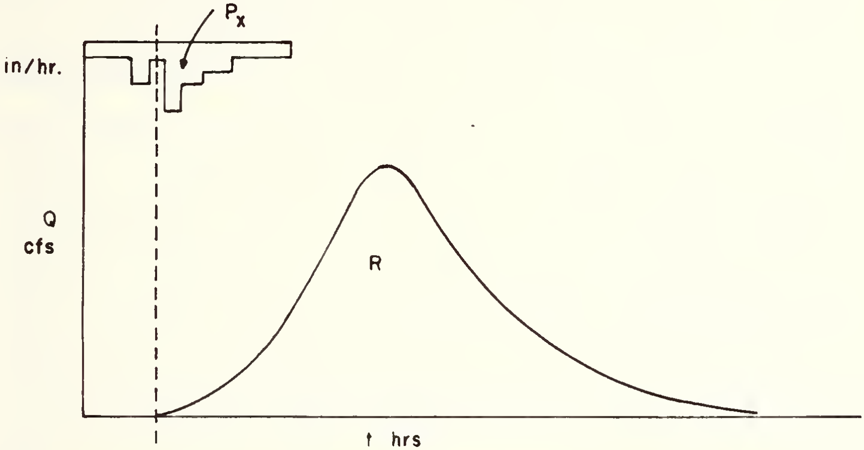


FIG. 6-1 STORM RAINFALL HYETOGRAPH AND RESULTING RUNOFF HYDROGRAPH

6.3 Evaluation of Total Runoff and Runoff Coefficient

The values of the runoff coefficient as described in the previous section were determined for the various storms on the watersheds studied. Since these watersheds studied were small and the rain gages used for estimating the rainfall were not closely and evenly distributed, the true hyetograph and average precipitation for a given storm over a given watershed could not be determined accurately. Consequently, the derived values of runoff coefficients r were not taken as a fixed constant, but rather as falling within a range, say 0.2 - 0.4 or 0.5 - 0.7, chosen so that it gives a reasonable estimate of the true value. Studying the general soil regions of Indiana and their subsoil permeability, it is found that the runoff coefficient is correlated to the permeability of the soil. The relation obtained between the runoff coefficient, type of soil and permeability is shown in Table 6-1. Since the relation was found to be logical and consistent, the runoff coefficient can be estimated from the knowledge of the soil type of the watershed. Hence, by locating a given watershed on the soil map, the runoff coefficient can be readily determined. Table 6-2 lists the recommended runoff coefficients for various types of soil for the runoff design of small watersheds in Indiana.

The design runoff can be computed by the formula:

$$R = r \cdot P_x \qquad (6\text{-}2)$$

Where R is the design runoff, in inches

r is the runoff coefficient

P_x is the rainfall depth that occurred after the beginning of runoff in inches

For design purposes it is usually assumed that the ground is saturated and that depression storage is filled at the beginning of rainfall. Under this assumption, the runoff starts at the same time as the rainfall and the rainfall P_x

occurring after the start of runoff is the same as the total rainfall P.

Values of the total rainfall P can be estimated from Figures 3-3 and 3-4.

Table 6-1

The Runoff Coefficient, Type of Soil, and
Degree of Permeability of Subsoil

Watershed number	Runoff Coefficient	Type of soil**	Degree of Permeability***
1	0.6 - 0.8	E	Moderately
2	0.6 - 0.8	L	Slowly
3	0.8 - 1.0	J	Very slowly
4	0.8 - 1.0	L	Slowly
5	0.7 - 0.8	E	Moderately
8	0.5 - 0.6	E	Moderately
9	0.5 - 0.7	E	Moderately
10	0.6 - 0.7	E	Moderately
12	0.3 - 0.4	A, F	Very and very slowly
13	0.4 - 0.6	G, I	Moderately and very slowly
14	0.4 - 0.5	A, F	Very and very slowly
16	0.8 - 1.0	J	Very slowly
17	0.2 - 0.3	A, F	Very and very slowly
19	0.8 - 1.0	J	Very slowly
20	0.5 - 0.7	G	Moderately and slowly
21	0.5 - 0.7	F	Very slowly
22	0.5 - 0.7	M, L, I	Moderately and slowly, very slowly

** Refer to Figure (3-5)
*** Refer to Table 3-5

Table 6-2

Recommended Runoff Coefficients
for
Various Types of Indiana Soils

Type of Soil	Runoff Coefficient
A, H	0.30
D, H, O	0.50
C, E, G, M, P	0.70
K, L, N	0.80
B, I, J	1.00
F	0.50 - 0.80*

*The F type of soil is as slowly permeable as types B, I, and J, but the losses due to depression storage are quite different. There are swamps and lakes in this region where the amount of runoff passing through the outlet is decreased.

7. DESIGN EXAMPLES

7.1 Determination of Peak Discharge (25 year)

From the studies in chapter 4, the procedure for peak discharge determination is as follows:

1. Peak discharge determination by the simple formula.

The watershed is delineated on a topographic map from which the area (A) in square miles, and the slope (S) in feet per 10,000 feet are determined. The 25-year peak discharge is obtained by introducing the values of the watershed characteristics A and S into formula (4-1), or by means of the working chart of Fig. 4-1.

2. Peak discharge determination by the extended formula.

The watershed is delineated on a topographic map from which the following qualitities are determined: the watershed area (A) in square miles, the mean relief (H) in feet, the main stream slope S in feet per 10,000 feet and the watershed shape factor (F). The watershed is also delineated on the drainage map from which the drainage density (D) in miles per square miles is obtained. The 25-year peak discharge is obtained by introducing the values of the watershed characteristics A, H, D, S, and F into formula (4-2) or by means of the working chart of Fig. 4-2.

Two examples illustrating the use of formulas (4-1) and (4-2) and the working charts, Figures (4-1) and (4-2), are as follows:

a. Simple formula for peak discharge determination.

Watershed No. 34

Watershed characteristics

Watershed area	(A)	156 square miles
Main stream slope	(S)	10.68 ft/10000 ft

Using the simple formula, the 25-year peak discharge can be obtained from the working chart, Fig. 4-1, following the dotted line,

$$Q = 13,300 \text{ cfs}$$

Using the simple formula without the use of chart

$$Q = 0.000\,783\ A^{2.63}\ S^{1.54} = 17{,}900 \text{ cfs}$$

b. Extended formula for peak discharge determination

Watershed No. 29

Watershed characteristics

Watershed area	(A)	125 square miles
Mean relief	(H)	84.7 feet
Drainage Density	(D)	4.50 mi/sq. mi.
Main stream slope	(S)	6.05 ft/10000 ft
Watershed shape factor	(F)	1.9[illegible]

Using the extended formula, the 25-year peak discharge can be obtained from the working chart, Fig. 4-2, following the dotted line,

$$Q = 2{,}500 \text{ cfs}$$

Using the extended formula without the use of the chart,

$$Q = 0.0718\ A^{0.914}\ H^{0.804}\ S^{0.537}\ D^{0.8[illegible]9}\ F^{0.436}$$
$$= 2514 \text{ cfs.}$$

The peak discharge of other frequencies may be determined from Fig. 4-3.

7.2 Procedures for Design Hydrograph Determination

From the studies in Chapter 5, the general procedure for the design hydrograph determination can be outlined as follows:

1. Determination of watershed characteristics

The delineation of the watershed on a topographic map and the determination of the watershed area in square miles (A), the length of main stream in miles (L), and the slope of the main stream in ft/10,000 ft. (S) are the first steps in the hydrograph design.

2. Determination of the hydrograph parameters t_p and K_1

The time to peak t_p and the storage coefficient K_1 are determined from the multiple correlation diagrams, Figures 5-6 and 5-7 or calculated from the regression formulas, Eqs. 5-6 and 5-7.

3. Determination of the shape of the instantaneous hydrograph

The ratio K_1/t_p is calculated. Using this value, the hydrograph parameter n is found from Figure 5-1. The shape of hydrograph is then determined using this value of n and Figure 5-2 or Table 5-2. A dimensionless short duration hydrograph may then be plotted.

4. Determination of the runoff coefficient

The given watershed is located on the soil map, Figure 3-5 and the runoff coefficient is selected by reference to Table 6-2.

5. Determination of design rainfall

As discussed in chapter 5 the short duration hydrograph is used as a good approximation of the design hydrograph. The duration of this hydrograph is of the order of 0.1 t_p and it was adopted as the design hydrograph because it gives higher peaks than hydrographs of longer duration. The correct application of this hydrograph to a design rainfall requires the generation of a hyetograph of design rainfall having a time interval equal to the duration of the hydrograph. The summation of the runoff produced by each of the increments in the hyetograph yields the hydrograph corresponding to the design rainfall as discussed in section 5-4. Alternatively a unit hydrograph of longer duration may be derived from the short duration hydrograph and then the runoff hydrograph is obtained by multiplying the ordinates of the unit hydrograph by the amount of rainfall corresponding to the longer duration, as discussed in section 5-4.

Because of uncertainties in the values of t_p and K_1 and the resulting possible variations in the shape of the hydrograph, an alternative simple semi-empirical method was developed for the determination of design rainfall. In this method, the design rainfall is taken as the rainfall obtained for a duration equal to the time to peak t_p of the short duration hydrograph or to six hours whichever is the larger. This design rainfall is then taken to be applicable to the hydrograph despite the fact that its duration is only $0.1\ t_p$. This method tends to overestimate the values of the runoff and, in particular, the peak discharge; but, in view of the uncertainties involved, it is considered to be a safe conservative procedure.

The procedure is first to use Equation 5-6 or Fig. 5-6 to estimate the value of t_p, Fig. 3-3 is then used to estimate the six hours rainfall expected with a return period of 25 years (or Fig. 3-4 for a return period of 50 years).

If the time to peak is larger than 6 hours, Table 3-4 is used to obtain a value of the design rainfall. If the time to peak is less than 6 hours, the value obtained from Fig. 3-3 (or 3-4) is taken as the design rainfall.

As discussed in section 6.3, the design rainfall is considered to occur with a condition of saturated ground so that the amount determined may be taken as equal to the quantity P_x to which the runoff coefficient can be applied.

6. Determination of total runoff

The total runoff can be determined from the design rainfall by Equation 6-2

$$R = P_x\ r$$

that is, the design rainfall times the runoff coefficient where both P_x and R are expressed in inches.

7. Computation of maximum discharge

Using the known values of t_p and R the maximum discharge can be computed from Equation 5-8 or from the numerical values given in Table 5-4 and plotted in Fig. 5-5. The value of Q_p obtained may be used as an estimate of peak discharge or as a basis for constructing the design hydrograph.

8. Plotting the storm hydrograph

From the dimensionless hydrograph, the time to peak t_p, and the maximum discharge Q_p, the short duration hydrograph can be plotted, which for small watersheds may be taken as the runoff hydrograph. Figure 7-1 is a sketch diagram showing the sequence of the steps employed to obtain the design hydrograph.

An example of the computation of a design hydrograph is shown as follows:

Watershed: Pleasant Run at Arlington Avenue, Indianapolis, Indiana.

Watershed characteristics:	Drainage area (A)	7.57 square miles
	Length of main stream (L)	3.6 miles
	Mean slope of main stream (S)	32.4 ft/10000 ft
Hydrograph	From Figure 5-6	t_p = 5.8 hours
	From Figure 5-7	K_1 = 4.7 hours

Hydrograph parameter n:
Since $K_1/t_p = 0.81$, from Figure 5-2, n = 5.

From the Table 5-4 ~~or~~ from Fig. 5-5.

$$\frac{Q_p t_p}{640\ AR} = 0.781$$

Design rainfall, Figure 5-1
25-year, 6-hour rainfall $\qquad P_x$ = 3.5 inches

Design runoff:

From Figure 3-5 and Table 6-2 Runoff coefficient r = 0.70

Runoff R = 0.7 x 3/5 = 2.45 inches

Maximum discharge Q_p

$$Q_p = \frac{1.67 \times 640 \times 2.45}{5.8} \times 0.730$$

= 1640 cfs.

Design Hydrograph:

Q_p = 1640 cfs, t_p = 5.8 hours n = 5

Table 7-1

Design Hydrograph, Pleasant Run at Arlington Avenue, Indianapolis

Dimensionless hydrograph*		Storm hydrograph	
t/t_p	Q/Q_p	t (hr.)	Q (cfs)
0.2	3.[illegible]5	1.16	65
0.4	[illegible]	2.32	[illegible]
0.6	[illegible].40	3.48	1,057
0.8	91.[illegible]0	[illegible]	1,49[illegible]
1.0	100.00	[illegible]	1,640
1.2	93.[illegible]0	6.96	1,530
1.4	77.70	8.13	1,274
1.6	[illegible]9.60	9.[illegible]0	976
1.8	[illegible]3.00	10.40	70[illegible]
2.0	29.40	11.60	482
3.0	2.[illegible]	17.40	[illegible]
4.0	[illegible].34	[illegible]	[illegible]

*The dimensionless hydrograph can be obtained from Figure 5-2

The derived design hydrograph is shown on Fig. 7-2

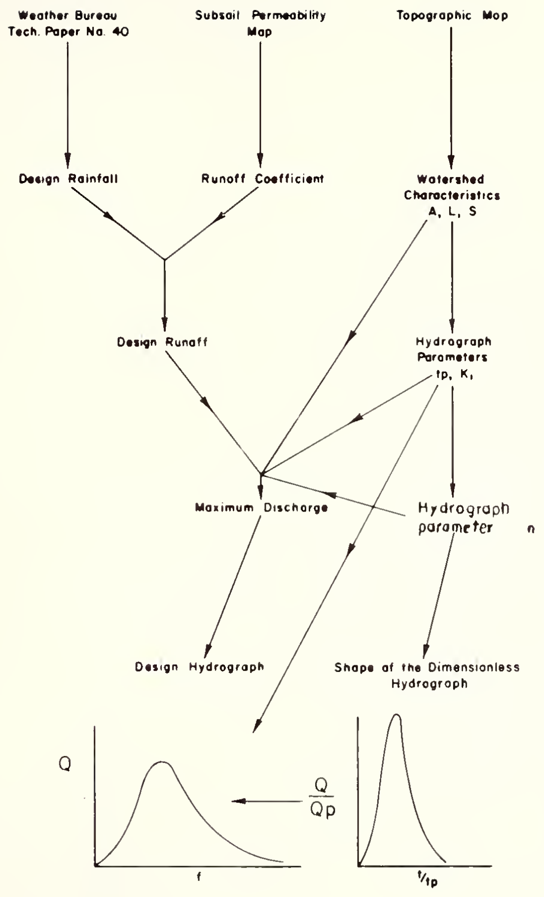


FIG 7-1 SEQUENCE OF COMPUTATIONS TO DESIGN STORM HYDROGRAPH FOR SMALL WATERSHED

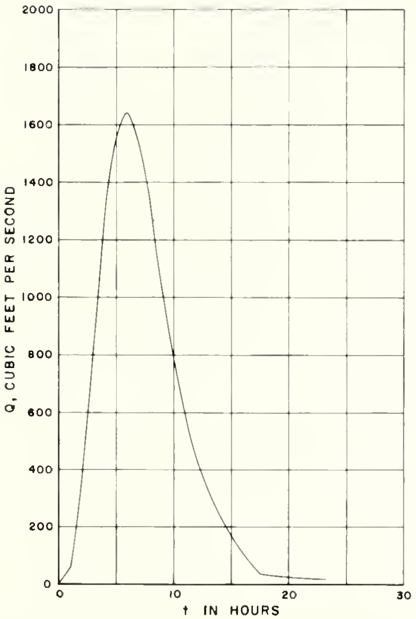


FIG. 7-2 DERIVED DESIGN HYDROGRAPH, PLEASANT RUN AT ARLINGTON AVENUE, INDIANAPOLIS

7.3 Comparison of Peak Discharge Determined from the Design Hydrograph with Results of Frequency Analysis.

As a check on the reliability of the results obtained by the hydrograph, some values of the peak discharge Q_p computed by this method were compared to actual values of the 25-year peak discharge Q_m as determined from the frequency analysis plot on probability paper. The results obtained for 7 small watersheds are shown in table 7-2; the values of Q_m were copied from table 4-1.

Table 7-2 shows that the maximum discharge obtained by these two methods agree quite well, except for one or two cases in which the disagreement may be due to poor estimates of the hydrograph parameters or of the runoff. In general, however, the method developed here seems to give results which are adequate for the design of hydrograph for small ungaged watersheds.

Table 7-2

Comparison of peak discharges from design hydrograph and frequency analysis

Watershed No.	Area sq. mi.	25-year peak discharge: design hydrograph (cfs) Q_p	Frequency study (cfs) Q_m	Deviation
[illegible]	[illegible]	[illegible]	[illegible]	320
[illegible]	[illegible]	[illegible]	[illegible]	900
[illegible]	[illegible]	[illegible]	[illegible]	560
19	[illegible]	[illegible]	20,500	4,700
[illegible]	[illegible]	19,400	12,900	6,500
[illegible]	[illegible]	2,400	1,630	770
22	100.0	11,800	11,800	0

Mean deviation 1,970 cfs

The mean deviation of the peak discharges from the design hydrograph and from the frequency analysis for the seven small watersheds studied is 1,970 cfs. It is of the same order of magnitude as the mean deviation using the extended formula, which is 2,040 cfs. It can be easily seen from the Table 7-2 that the peak discharge calculated from the hydrograph design, in general, is higher than those from the frequency analysis, this agrees with the theory of instantaneous unit hydrograph which gives an upper limit of the estimation.

SUMMARY AND CONCLUSIONS

1. In the frequency analysis all available past observations of annual peak discharge were plotted on a probability paper using Gumbel's extreme value theory. A straight line of best fit was passed through the plotted point as predicted by the theory of extreme values. Expected floods with different frequencies were obtained by extending the straight line. Table 4-1 gives the predicted flood of 25, 50, 75 and 100 years of frequency for 32 gaged watersheds in Indiana.

2. The geomorphological characteristics of the small watersheds are considered to be the dominant factors which affect the peak discharge. The application of the multiple correlation technique to the study of the relationship between the 25 year peak discharge and the geomorpholical watershed characteristics resulted in two equations for the indirect determination of the 25 year peak discharge. The first equation is based on two watershed characteristics and was called the simple formula. The second equation is based on five watershed characteristics and was called the extended formula. These correlations are based on the assumption that the climatological and geological conditions are reasonably homogeneous throughout the state. The geomorphological factors considered significant are: the watershed area, the drainage density, the mean relief of watershed, the main stream slope, and the shape factor of the watershed. The extended formula uses these five geomorphological characteristics and the simple formula used only the area and the main stream slope.

3. A working chart (Fig. 4-2) was prepared to obtain the 25-year peak discharge directly from the five watershed characteristics, for areas from 50 up to 250 square miles. As shown in the example of article 7-1 the design engineers may use the design chart to estimate the 25-year peak

discharge with good accuracy. The peak discharge with other frequencies may be obtained from Fig. 4-3.

4. The simple formula contains only two geomorphological factors: A and S. It is suggested as a first approximation. A working chart for this formula is given in Fig. 4-1. It is less accurate than the extended formula, but is simple and rapid for the peak discharge determination.

5. Since the small watersheds used in the peak discharge determination range in area from 50-250 square miles, the use of the formulas developed herein is recommended only for areas in this range.

6. The study of the hydrograph is based on fundamental concepts of hydrology. The parameters of the theoretical hydrograph of short duration are correlated statistically to three watershed characteristics. Equations were derived for the time to peak (t_p) and for the recession constant (K_1) of the short duration unit hydrograph in terms of the watershed area A, the main stream length L and slope S. From these two quantities t_p and K_1, the value of the hydrograph parameter n can be determined. The value of n completely specifies the shape of the dimensionless hydrograph of short duration.

7. As mentioned in chapter 5, the use of the shape of the short duration hydrograph yields a good estimation of the runoff hydrograph for small watersheds. It is also a safe design since short duration hydrograph gives higher peak then the hydrograph with longer durations.

8. The indirect determination of t_p and K_1 by means of the watershed characteristics A, L and S in formulas (5-6) and (5-7) is only a statistical correlation indicating the relationship among them for the studied watersheds. Other methods may be used to determine t_p and K_1.

9. As mentioned before, the design runoff is based on the design rainfall and the runoff coefficient corresponding to the watershed location. Obviously, the worth of the derived design hydrograph hinges in a large measure upon the estimates of the value of runoff. Since the runoff coefficient is not a fixed value, the estimate of the total runoff may well vary with the judgment of the individual. The suggested runoff coefficients in Table 6-2 are considered to be conservative.

10. For convenience in practical engineering design, the hydrograph study has been directed toward making the design procedure as simple as possible. Most of the required data can be obtained from topographic maps, and from the working charts and tables presented herein.

11. Since the small watersheds used in the hydrograph study range in area from 2.86 to 100 square miles, the use of the procedures developed herein is recommended only for watersheds between 3 and 100 square miles.

12. With reference to the comparison of the peak discharge determined from the design hydrographs with the results of the frequency analysis (Art. 7-3), it should be remarked that the maximum annual flow Q_m determined by the frequency analysis includes the base flow, whereas the value of the peak discharge Q_p determined by the hydrograph method does not include base flow. However, on one hand, the base flow for small watersheds is usually very small, and, on the other hand, the instantaneous unit hydrograph method gives an upper limit of the peak discharge. Consequently, the two errors tend to compensate each other.

13. Strictly speaking, the 25-year storm does not necessarily result in the 25-year peak runoff, due to variations in antecedent moisture and other factors. However, for small watersheds, this variation is smaller then for large watersheds. In addition, the hydrograph method assumes that the soil is saturated at the beginning of the rainfall. It is thus justifiable to compare the 25-year peak flood obtained from the frequency analysis to the peak discharge resulting from the 25-year storm calculated by the hydrograph method.

(6) Bice ditch near South Marion, Ind

Location.--Lat 40°52', long 87°06', on line between secs 15 and 22, T 28 N., R 6W., on left bank at upstream side of bridge on State Highway 16, 2 miles upstream from Big Slough Creek, 3 miles southeast of South Marion, and 5 miles southeast of Rensselaer

Drainage area.--22.6 sq mi

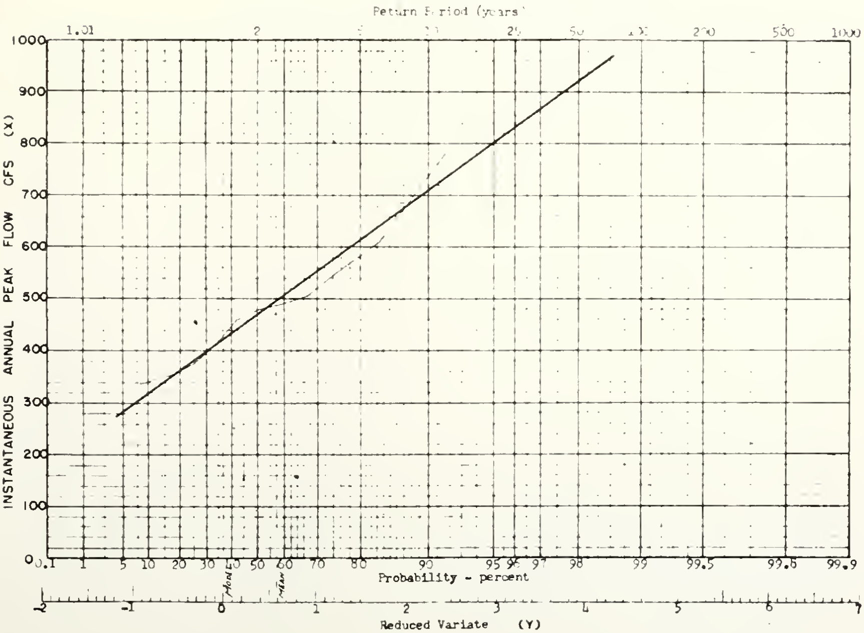
Gage.--Nonrecording gage Dec 31, 1948, to Aug 4, 1955; recording gage thereafter. Datum of gage is 653.30 ft above mean sea level, datum of 1929

Stage discharge relation.--Defined by current-meter measurements

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1949	Feb 15, 1949	9.09	410	1955	June 11, 1955	10.16	353
1950	July 19, 1950	10.06	490	1956	Apr 29, 1956	10.75	504
1951	July 9, 1951	11.43	610	1957	July 13, 1957	10.36	458
1952	June 14, 1952	10.80	556	1958	June 13, 1958		780
1953	July 5, 1953	8.65	374	1959	Feb 10, 1959		480
1954	June 22, 1954	8.12	339				

BICE DITCH NEAR SOUTH MARION, INDIANA



(7) Iroquois River at Rosebud, Ind

Location --Lat 41°02', long 87°11', in SW 1/4 sec. 24, T 30 N., R 7 W., 100 ft downstream from bridge on county road, half a mile north of Rosebud, half a mile downstream from confluence of Swain and Dexter ditches 1.5 miles upstream from Davidson ditch, and 2 miles east of Parr.

Drainage area --30 3 sq mi

Gage.--Nonrecording gage July 22, 1948, to Sept 30, 1953; recording gage thereafter. Datum of gage is 661 47 ft above mean sea level, datum of 1929.

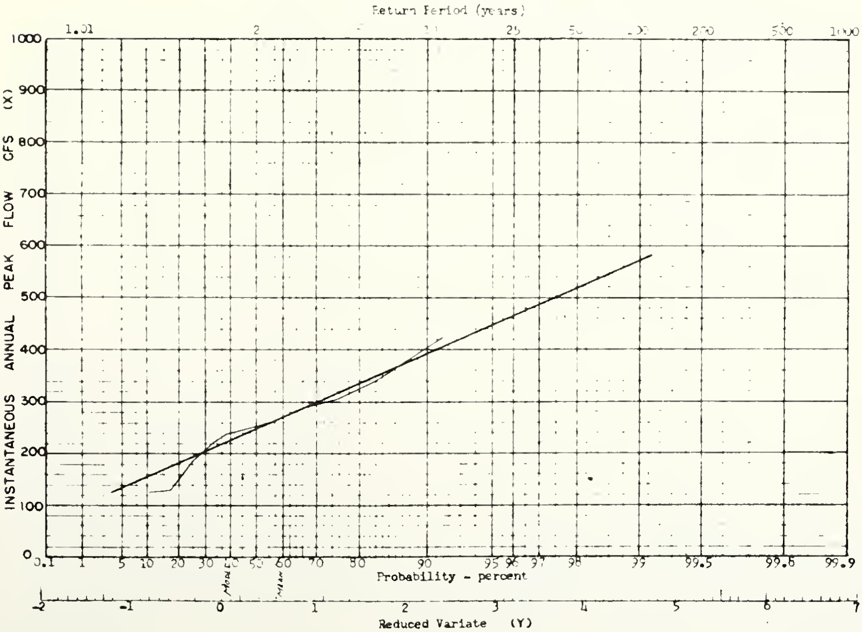
Stage-discharge relation.--Defined by current meter measurements below 330 cfs

Flood stage --10 ft

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1949	Feb 15, 1949	6 15	254	1955	Jan 6, 1955	4 84	126
1950	Apr 4, 1950	8 3	432	1956	Feb 29, 1956	6 65	225
1951	July 9, 1951	7 2	235	1957	Apr 28, 1957	7 90	290
1952	Apr 23, 1952	7 3	263	1958	June 10, 1958		308
1953	Mar 15, 1953	5 75	[illegible]	1959	Feb [illegible], 1959		343
1954	Mar 25, 1954	4 59	[illegible]				

IROQUOIS RIVER AT ROSEBUD, INDIANA



(10) Cicero Creek near Arcadia

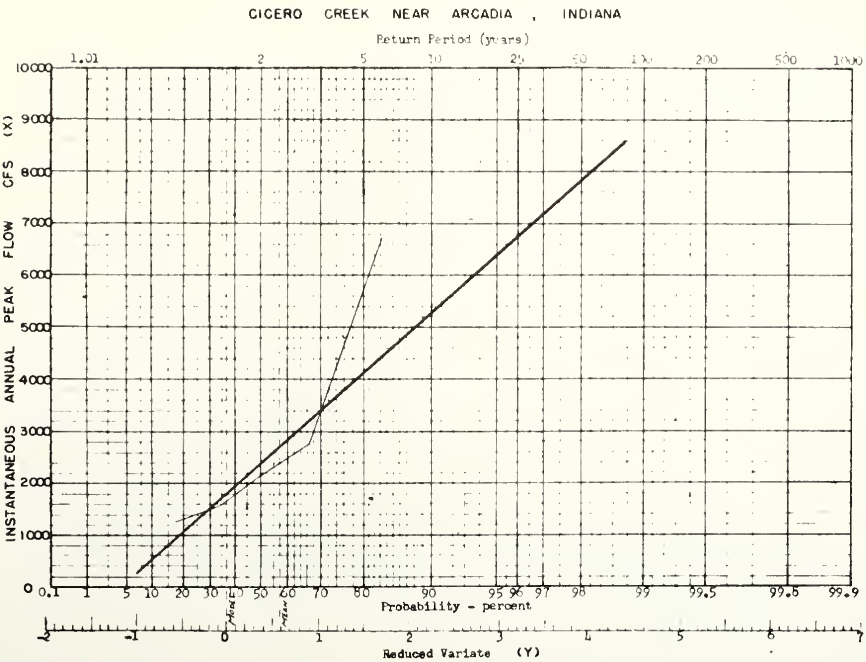
Location --Lat 40°11', long 86°00', on line between secs. 18 and 19, T. 20 N., R. 5E., on left bank on downstream side of county bridge, 1¼ miles east of Arcadia, Hamilton County, and 5 miles upstream from little Cicero Creek

Drainage area --131 sq. mi

Gage -- Water stage recorder. Datum of gage is 815.12 ft above mean sea level Datum of 1929. Prior to Dec. 7, 1955, wire weight gage at same site and datum.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1955	July 16, 1955		1,280	1958	June 15, 1958		2,740
1956	July 21, 1956		1,540	1959	Feb. 11, 1959		2,170
1957	June 29, 1957		6,720				



(11) Carpenter Creek at Egypt, Ind

Location --Lat 40°52', long 87°12 , on line between SW¼ sec. 15 and NW¼ sec 22 T 28 N , R. 7 W , on left bank on downstream side of bridge on State Highway 16 2 3/4 miles upstream from mouth, and 4 miles southwest of Collegeville

Drainage Area --48 1 sq mi

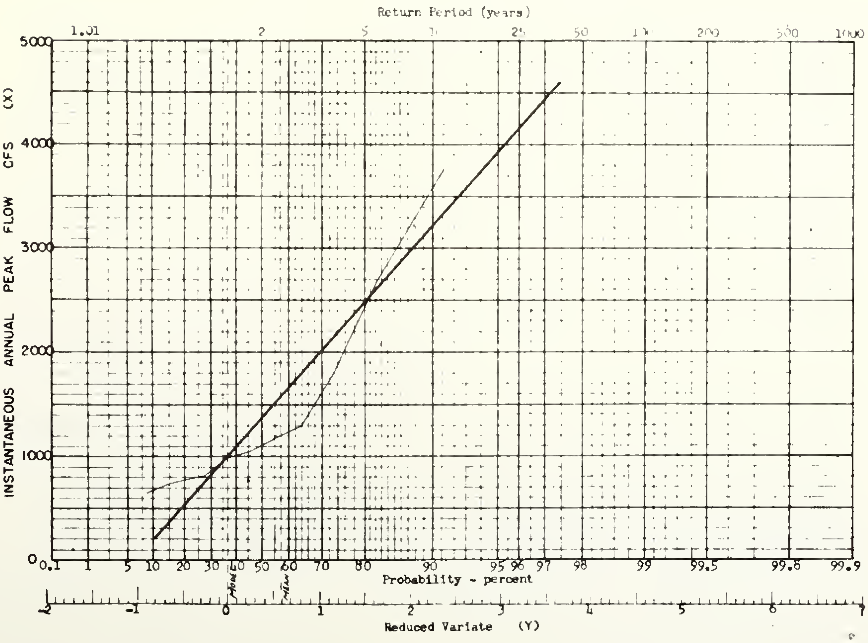
Gage Nonrecording gage July 26, 1948, to Dec 31, 1951, and Oct. 1, 1952, to Sept 5, 1955; recording gage since Sept. 6, 1955 Datum of gage is 640 37 above mean sea level, datum of 1929.

Stage discharge relation.--Defined by current-meter measurements

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1949	Feb 15, 1949	10 14	1 160	1955	June 8, 1955	9 80	984
1950	Apr 4, 1950	10 3	1,300	1956	Apr [illegible], 1956	9 93	1,040
1951	July 9, 1951	10 92	1,790	1957	July 13, 1957	9 42	810
1953	July 6, 1953	9 21	735	1958	June 10, 1958		3,720
1954	June 22, 1954	8 95	655	1959	[illegible] 10, 1959		2,690

CARPENTER CREEK AT EGYPT , INDIANA



(12) West Creek near Schneider, Ind

Location.--Lat 41°12'52", long 87°29'36", in NW 1/4 NE 1/4 sec. 19, T 32 N., R 9 W., on left bank at downstream side of county highway bridge, 1.2 miles upstream from Singleton ditch and 2 3/4 miles northwest of Schneider

Drainage area.--54.5 sq mi.

Gage.--Nonrecording gage July 29, 194[illegible], to Dec. 31, 1951, and Jan. 1, 1954, to June 10 1956; recording gage since June 11, 1956. Datum of gage is 627.86 ft above mean sea level, datum of 1929 (levels by Soil Conservation Service)

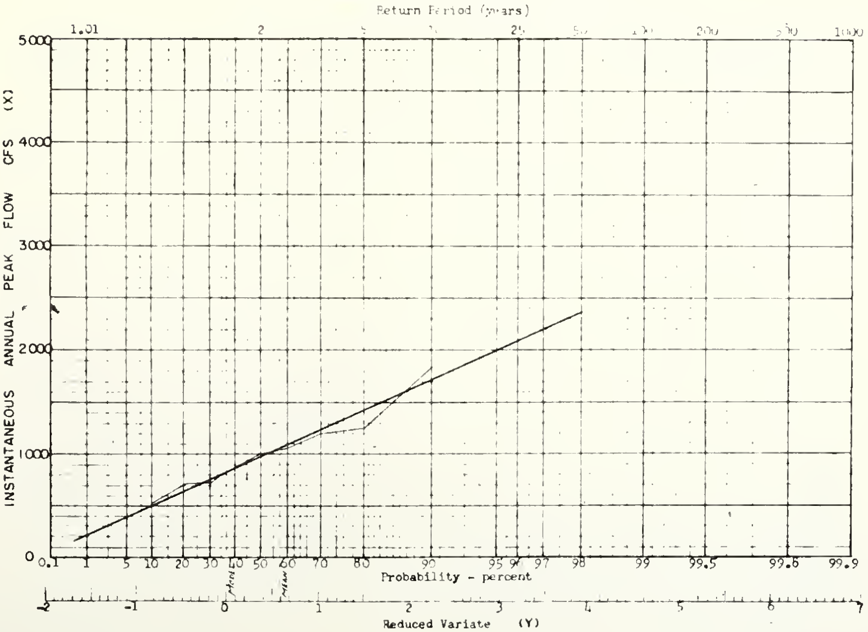
Stage discharge relation.--Defined by current-meter measurements

Flood stage [illegible]7 ft.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1949	Feb 1[illegible], 1949	4 58	50[illegible]	1956	Feb 25 1956	5 42	710
1950	Dec 22, 1949	6 56	1,[illegible]	1957	July 13, 1957	7 02	1,250
1951	Feb 19, 1951	5 5[illegible]	7[illegible]8	1958	June 9, 1958		794
1954	Mar 25, 1954	[illegible] 10	1,[illegible]00	1959	Apr 28, 1959		1,200
1955	Oct 10, 1954	8 [illegible]	1 840				

WEST CREEK NEAR SCHNEIDER, INDIANA



(14) Little Calumet River at Porter, Ind.

Location.--Lat 41°37'18", long 87°05'13", in NE 1/4 sec. 34, T. 37 N., R. 6 W., near center of span of downstream side of highway bridge, three-quarters of a mile northwest of Porter, and 4.5 miles upstream from Salt Creek.

Drainage area.--62.9 sq mi.

Gage.--Nonrecording gage May 5, 1945, to June 25, 1952; recording gage thereafter. Datum of gage is 603.48 ft above mean sea level, datum of 1929.

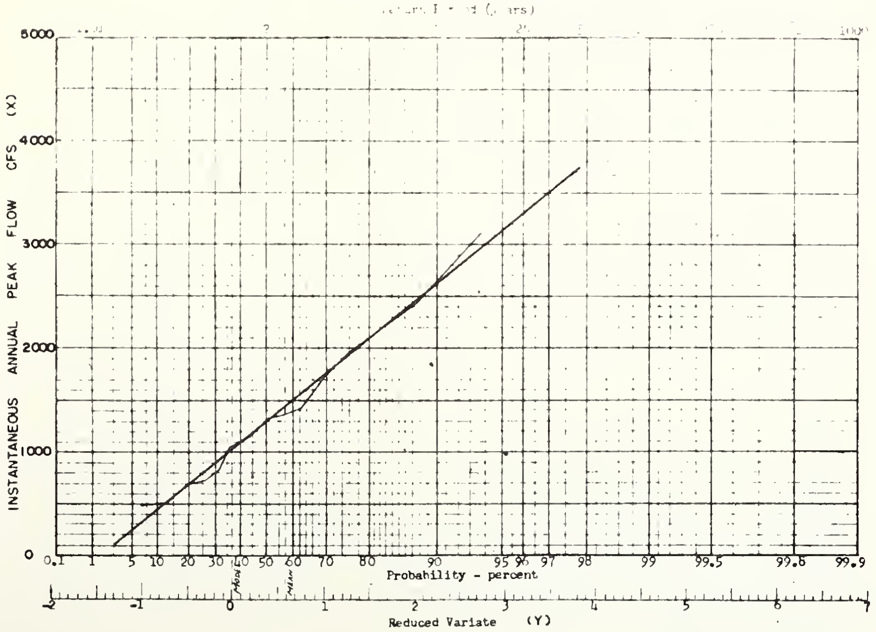
Stage-discharge relation.--Defined by current-meter measurements below 2,500 cfs. Rating subject to changes throughout range of stage.

Flood stage.--7 ft.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1945	June 28, 1945	9.88	2,440	1953	May 23, 1953	6.64	521
1946	June 13, 1946	6.99	715	1954	Apr 26, 1954	8.32	1,170
1947	Apr 5, 1947	9.42	2,140	1955	Oct. 10, 1954	11.66	3,110
1948	May 11, 1948	9.10	1,960	1956	Apr. 29, 1956	8.67	1,370
1949	May 20, 1949	6.88	690	1957	Apr. 27, 1957	7.65	848
1950	Dec 22, 1949	8.72	1,720	1958	Feb. 28, 1958	-	490
1951	May 11, 1951	8.11	1,360	1959	Apr. 28, 1959		1,420
1952	Nov. 14, 1951	7.92	1,060				

LITTLE CALUMET RIVER AT PORTER, INDIANA



(15) Hart ditch at Munster, Ind.

Location.--Lat 41°33'40", long 87°28'50", in N 1/2 sec. 20, T. 36 N., R. 9 W., on left bank at city limits of Munster, a quarter of a mile downstream from U. S. Highway 41, and 0.4 mile upstream from mouth.

Drainage area.--69.2 sq mi.

Gage.--Recording. Datum of gage is 591.21 ft above mean sea level, datum of 1929.

Stage discharge relation.--Defined by current-meter measurements. Dredging operations assumed to have occurred between April 1944 and April 1945, and subsequent filling have affected high-water rating. Backwater from Little Calumet River and possibly from overbank return effects stage at gage at times during periods of extremely high flow.

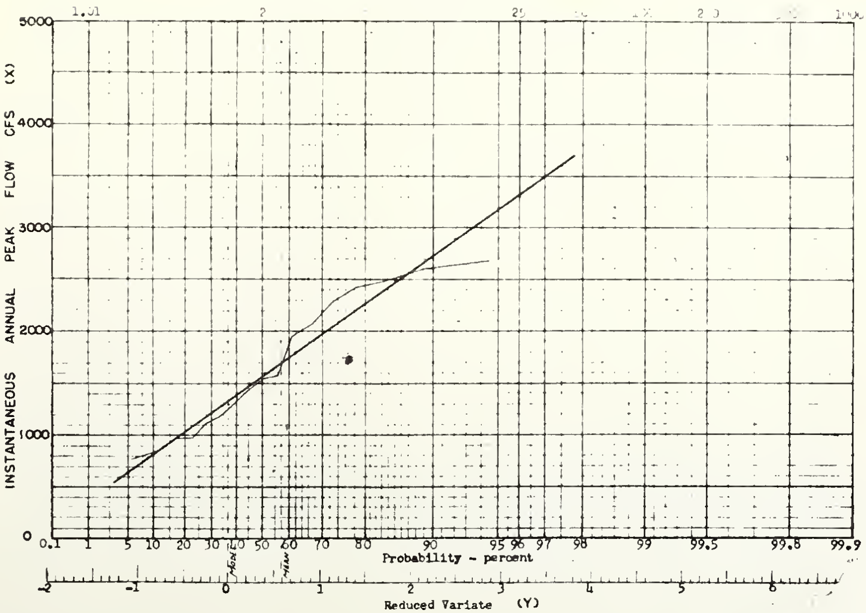
Flood Stage.--7 ft

Remarks.--Hart ditch is tributary to Little Calumet River. At this point low flow of Little Calumet River runs west into Calumet Sag Channel or into Lake Michigan through Grand Calumet River; floodflow at times runs east into channel storage or through Burns ditch to Lake Michigan.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1943	Mar. 16, 1943	6.95	2,280	1952	June 14, 1952	4.39	1,190
1944	Mar. 15, 1944	7.23	2,420	1953	Mar 15, 1953	3.84	960
1945	May 8, 1945	3.73	1,270	1954	Mar 25, 1954	4.25	1,110
1946	Jan. 6, 1946	2.88	780	1955	Oct. 11, 1954	7.83	2,600
1947	Apr. 6, 1947	6.17	2,490	1956	May 11, 1956	5.27	1,550
1948	May 11, 1948	5.60	1,950	1957	July 14, 1957	7.60	2,060
1949	Feb 13, 1949	3.00	850	1958	June 10, 1958		960
1950	Dec. 22, 1949	4.83	1,570	1959	Apr 28, 1959		2,670
1951	May 1, 1951	5.01	1,430				

HART DITCH AT MUNSTER, INDIANA



(17) Salt Creek near McCool, Ind

Location Lat 41°35'48", long 87°08'40", in SE¼ sec 6, T 36 N, R 6 W, on left bank on downstream side of highway bridge, 50 ft downstream from New York Central Railroad bridge, 1½ miles north of McCool, and 1 5 miles upstream from Little Calumet River

Drainage area --78 7 sq mi.

Gage --Nonrecording gage May 5, 1945, to July 24, 1955; recording gage thereafter Datum of gage is 594 10 ft above mean sea level, datum of 1929 (levels by Indiana Flood Control and Water Resources Commission).

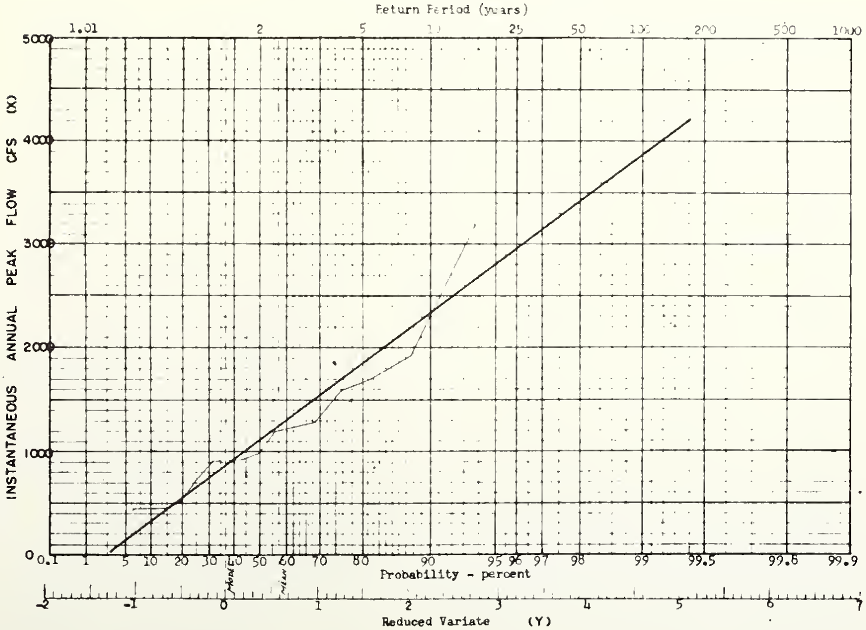
Stage discharge relation.--Defined by current-meter measurements below 2,300 cfs

Flood stage - 10 ft

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1945	June 29, 1945	10 58	990	1953	Mar 16, 1953	8 16	454
1946	June 13, 1946	11 27	1,280	1954	Mar 26, 1954	10 48	910
1947	Apr 5, 1947	11 83	1,580	1955	Oct 11, 1954	14 12	3,180
1948	May 11, 1948	12 3	1,910	1956	Apr. 29, 1956	11.26	1,280
1949	Feb 14, 1949	9 28	525	1957	Apr. 27, 1957	9 81	725
1950	Dec 22, 1949	12 02	1,700	1958	Nov 15, 1957		456
1951	May 11 1951	10 78	970	1959	Apr 28, 1959		1,200
1952	Nov 14, 1951	10 63	912				

SALT CREEK NEAR McCOOL, INDIANA



(18) Big Slough Creek near Collegeville, Ind

Location --Lat 40°53', long 87°09', in SW¼ NW¼ sec 7, T 28 N , R. 6 W , on right bank on downstream side of bridge on State highway 53, 1½ miles south of Collegeville, 2½ miles upstream from mouth, and 2 3/4 miles downstream from Rice ditch

Drainage area --84 1 sq mi

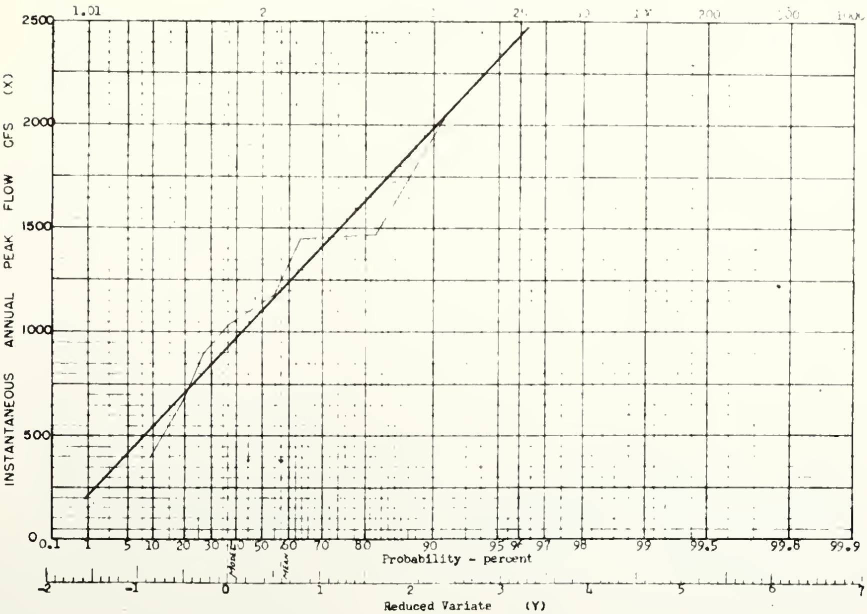
Gage --Nonrecording gage July 28, 1948, to Dec 31 1951, and Oct 1, 1952, to Aug 4 1955; recording gage since Aug. 5, 1955 Datum of gage is 637 75 ft above mean sea level, datum of 1929.

Stage-discharge relation. -Defined by current-meter measurements

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1913	March 1913	13 7		1954	June 22, 1954	8 84	390
1927	1927	12 5		1955	June 11, 1955	12 4	1,100
1949	[illegible] 15, 1949	11 26	880	1956	Apr 29, 1956	13 0	1,470
1950	Apr [illegible] 1950	[illegible]	[illegible]	1957	[illegible] 24, 1957	12 96	1,470
1951	July [illegible] 1951	[illegible]	[illegible]	1958	June 13, 1958		2,030
1953	Apr 1, 1953	[illegible]	[illegible]	1959	Apr 13 1959		1,030

BIG SLOUGH CREEK NEAR COLLEGEVILLE , INDIANA



(19) North Fork of Vernon Fork near Butlerville, Ind.

Location.—Lat 39°02'55", long 85°32'40", in SE¼ sec. 17, T. 7 N., R. 9 E., on left bank, 0.3 mile downstream from Muscatatuck State School dam, 1¾ miles downstream from Brush Creek, and 2 miles northwest of Butlerville.

Drainage area.—87.3 sq mi

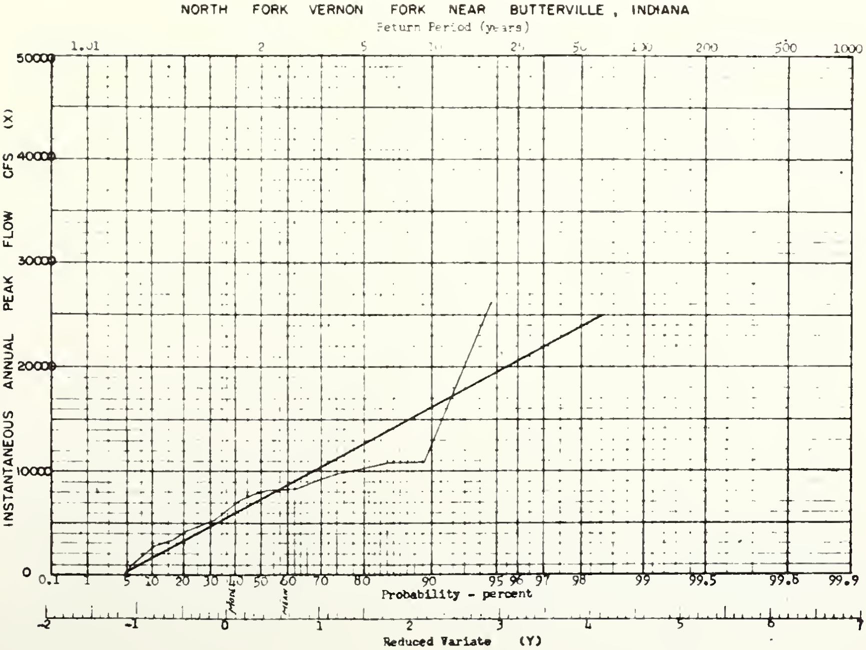
Gage.—Nonrecording gage Feb. 16, 1942, to Aug. 18, 1942; recording gage thereafter. Datum of gage is 669.40 ft above mean sea level, datum of 1929.

Stage-discharge relation.—Defined by current-meter measurements.

Flood stage.—11 ft.

Peak Stages and Instaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1942	Apr 9, 1942	8.94	2,560	1951	Nov. 20, 1950	15.98	8,030
1943	Mar. 16, 1943	17.79	9,910	1952	Jan 26, 1952	13.18	5,300
1944	Apr 11, 1944	12.63	4,780	1953	Mar. 4, 1953	10.34	3,260
1945	Mar 6, 1945	18.72	10,900	1954	Jan. 1, 1954	5.58	840
1946	Feb 13, 1946	15.95	8,030	1955	Feb. 27, 1955	12.05	4,300
1947	June 2, 1947	14.30	6,330	1956	May 28, 1956	16.23	8,330
1948	Mar 27, 1948	15.12	7,130	1957	May 22, 1957	17.04	9,080
1949	Jan 24, 1949	18.73	10,900	1958	July 22, 1958		7,730
1950	Jan 4, 1950	17.90	10,000	1959	Jan 21, 1959		26,200



(20) Clifty Creek at Hartsville, Ind.

Location.—Lat 39°16'25", long 85°42'10", in NW¼ sec. 36, T. 10 N., R. 7 E., at downstream side of left abutment of highway bridge, a quarter of a mile north of Hartsville, and 5 miles upstream from Duck Creek.

Drainage area.—88.8 sq mi.

Gage.—Nonrecording gage Feb. 12, 1948, to Sept. 23, 1952; recording gage thereafter. Datum of gage is 677.34 ft above mean sea level, datum of 1929.

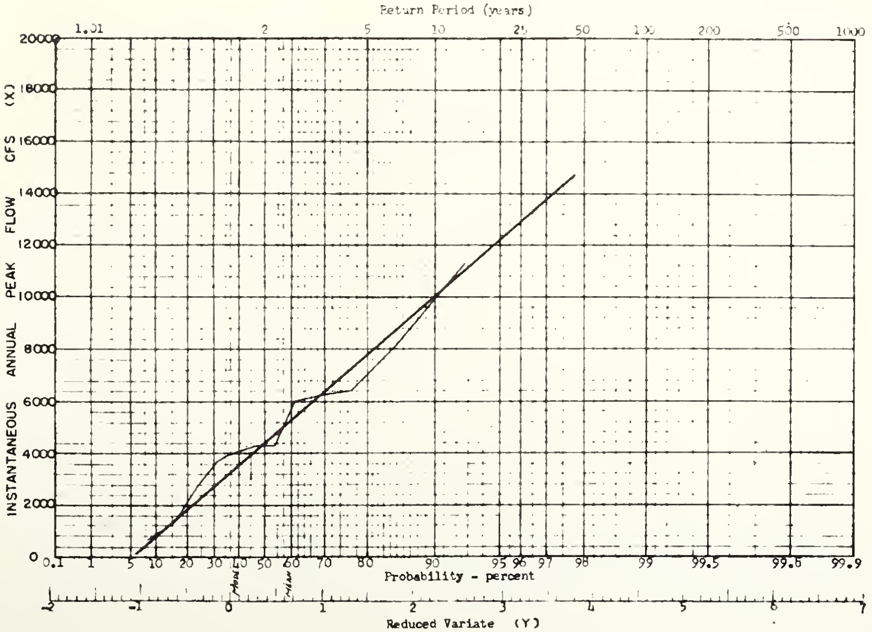
Stage-discharge relation.--Defined by current-meter measurements below 6,000 cfs.

Historical data.--Flood of 1913 on Clifty Creek reached a stage of about 3 ft higher than the McKinley (1897) flood according to a report in the Evening Republican of Columbus, Ind. dated Mar. 25, 1913. (The preceding statement was apparently for the Petersville area, about 6 miles downstream from Hartsville).

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1913	Mar. 25, 1913	25.1		1954	May 27, 1954	4.17	635
1948	Mar. 27, 1948	8.48	3,710	1955	July 8, 1955	6.24	1,760
1949	Jan 5, 1949	13.4	8,100	1956	June 22, 1956	11.10	5,890
1950	Jan 4, 1950	11.8	6,520	1957	July 4, 1957	9.28	4,270
1951	Nov. 20, 1950	8.9	3,910	1958	May 6, 1958		2,700
1952	Jan 26, 1952	11.5	6,250	1959	Jan. 21, 1959		11,300
1953	Mar. 4, 1953	5.57	1,370				

CLIFTY CREEK AT HARTSVILLE, INDIANA



(21) Cedar Creek at Auburn, Ind

Location.--Lat 41°21', long 85°03', in SW 1/4 sec. 29, T. 34 N., R. [illegible], near center of span on upstream side of Ninth Street Bridge in Auburn and 2 miles upstream from [illegible] ditch.

Drainage area.--93 sq mi., approximately.

Gage.--Nonrecording gage July 30, 1942, to Sept. 30, 1953; recording gage thereafter. Datum of gage is 847.14 ft above mean sea level (city of Auburn bench mark).

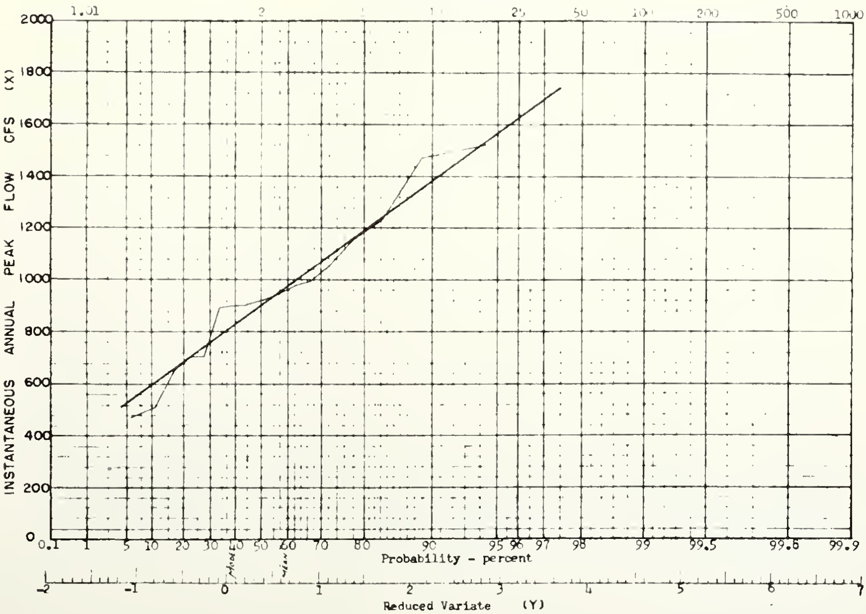
Stage-discharge relation.--Defined by current-meter measurements.

Flood stage.--8 ft.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1943	May [illegible], 1943	9.8	1,470	1952	[illegible] 11, 1952	8.45	900
1944	Apr. 12, 1944	9.3	1,230	1953	Mar. 4, 1953	5.80	471
1945	May 18, 1945	9.13	1,150	1954	[illegible] 25, 1954	7.57	707
1946	June 13, 1946	8.58	915	1955	Jan. 6, 1955	7.61	707
1947	Apr. 21, 1947	9.02	983	1956	[illegible], 1956	8.85	1,050
1948	Feb. 28, 1948	8.53	905	1957	[illegible] 6, 1957	6.89	651
1949	Feb. 16, 1949	7.21	995	1958	Dec. 20, 1957		540
1950	Apr. 5, 1950	9.[illegible]	1,520	1959	Feb. 14, 1959		890
1951	[illegible]	[illegible]	[illegible]				

CEDAR CREEK AT AUBURN, INDIANA



(22) Bean Blossom Creek at Dolan, Ind.

Location.--Lat 39°14'30", long 86°29'57", in SW¼ sec. 2, T. 9 N., R. 1 W., on downstream side of right pier of highway bridge at Dolan, 17.5 miles upstream from mouth.

Drainage area.--100 sq mi.

Gage.--Nonrecording gage Apr. 3, 1946, to Sept. 27, 1951; recording gage thereafter Datum of gage is 576.41 ft above mean sea level, unadjusted.

Gage-discharge relation.--Defined by current-meter measurements. Discharge adjusted for rate of change of stage above 5 ft. Only annual maximums adjusted prior to installation of recording gage.

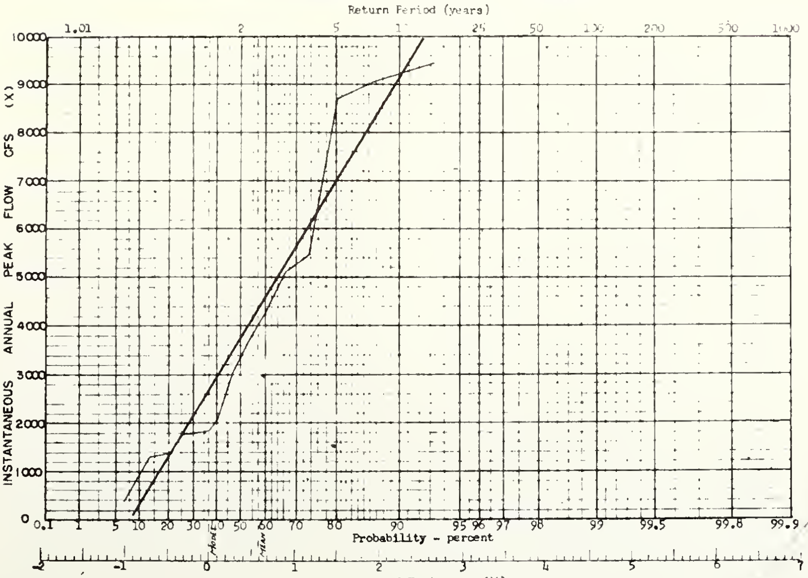
Flood stage.--15 ft.

Remarks.--Flow regulated since April 1953 by Bloomington Reservoir (capacity, 4,640,000,000 gallons) 7½ miles upstream; peak discharges probably not materially affected.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1946	May 16, 1946	13.0	1,830	1953	Mar 4, 1953	11.07	1,320
1947	June 2, 1947	17.8	9,420	1954	May 2, 1954	5.45	361
1948	Mar.27, 1948	13.5	2,110	1955	Apr.13, 1955	11.63	1,390
1949	Jan. 5, 1949	17.9	9,060	1956	May 28, 1956	12.93	1,740
1950	Jan. 4, 1950	17.75	8,740	1957	May 22, 1957	15.78	4,270
1951	Jan.21, 1951	15.50	3,700	1958	June 14, 1958		3,040
1952	May 24, 1952	16.12	5,100	1959	Jan. 21, 1959		5,480

BEAN BLOSSOM CREEK AT DOLAN, INDIANA



(23) Pigeon Creek at Hogback Lake Outlet, near Angola, Ind.

Location.--Lat 41°37'24", long 85°05'4.", in NE 1/4 NW 1/4 sec. 36, T. 37 N., R. 12 E., on right bank 200 ft north of lake outlet, 2 miles southeast of Flint, and 5.1 miles west of Angola.

Drainage area.--102 sq mi, 105 sq mi prior to October 1947.

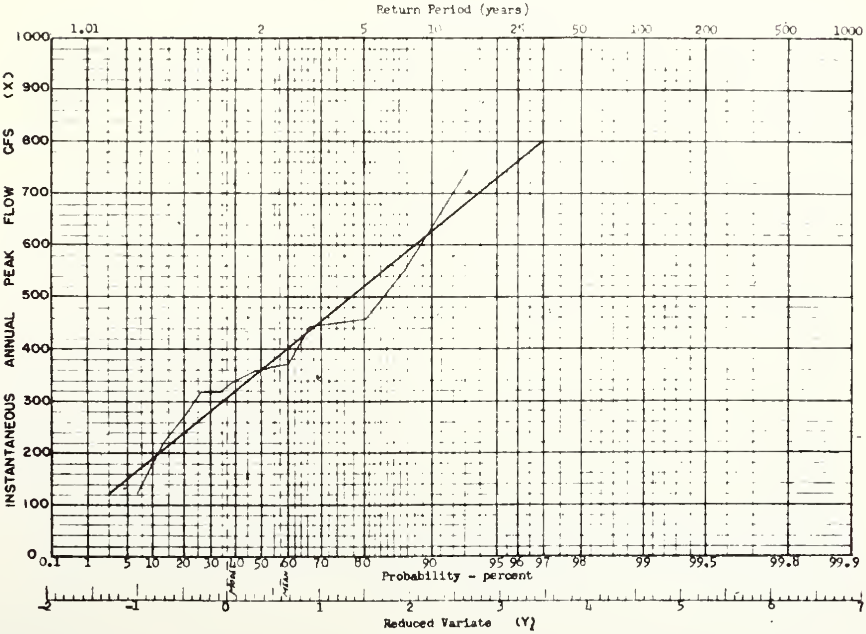
Gage.--Nonrecording gage Oct. 16, 1945, to Aug. 3, 1953; recording gage thereafter. Prior to Oct. 1, 1947, at site 1 1/2 miles downstream at different datum. Oct. 1, 1947, to Aug. 3, 1953, at site 600 ft downstream at present datum. Datum of present gage is 940.00 ft above mean sea level, datum of 1929.

Stage-discharge relation.--Defined by current-meter measurements below 240 cfs at former site and by current-meter measurements at present site.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1946	Feb. 19, 1946	-	220	1953	Mar 19, 1953	9.30	122
1947	Apr. 24, 1947	10.71	458	1954	Mar. 30, 1954	11.31	317
1948	Mar 2, 1948	11.79	355	1955	Oct. 17, 1954	11.54	339
1949	Feb. 19, 1949	11.93	366	1956	May 4, 1956	13.39	548
1950	Apr 8, 1950	14.95	744	1957	Apr. 14, 1957	11.29	317
1951	Feb. 24, 1951	12.50	448	1958	Sept.21, 1957		274
1952	Jan 21, 1952	11.85	370	1959	Feb.17-19,1959		442

PIGEON CREEK AT HAGBACK LAKE OUTLET NEAR ANGOLA , INDIANA



(24) Young Creek near Edinburg, Ind

Location.--Lat 39°25'08", long 86°00'18", in SW 1/4 sec 5, T 11 N., R 5 E., on left bank, on upstream side of highway bridge half a mile southwest of Amity, 2 miles upstream from mouth, and 5 miles northwest of Edinburg

Drainage area.- 109 sq mi

Gage.--Nonrecording gage Dec 7, 1942, to June 29, 1955; recording gage thereafter. Datum of gage is 670.2 ft above mean sea level, datum of 1929.

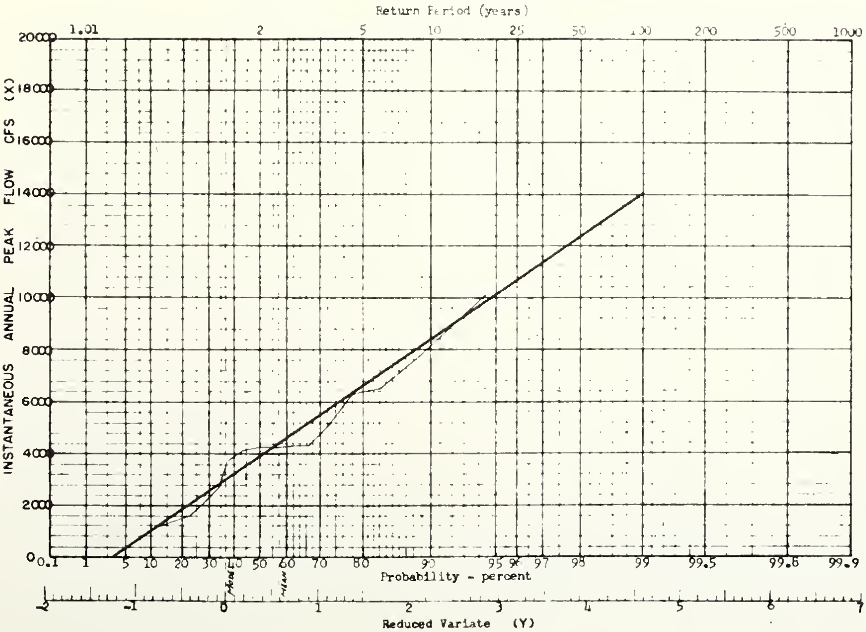
Stage-discharge relation.--Defined by current-meter measurements below 7,000 cfs and by contracted-opening measurement at 10,700 cfs

Flood stage.- 7 ft

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1943	Mar 19, 1943	10.40	3,700	1952	Jan 27, 1952	13.4	10,700
1944	Apr 11, 1944	11.00	4,290	1953	Mar 4, 1953	8.37	2,080
1945	Mar 6, 1945	11.00	4,290	1954	Jan 27, 1954	3.27	443
1946	May 16, 1946	9.0	2,510	1955	May 28, 1955	6.2	1,110
1947	June 2, 1947	11.12	4,390	1956	Nov 16, 1955	12.20	7,790
1948	Mar 27, 1948	7.68	1,650	1957	July 5, 1957	11.62	6,510
1949	Jan 5, 1949	11.9	5,190	1958	June 11, 1958		4,350
1950	Jan 4, 1950	10.8	4,090	1959	Jan 21, 1959		6,270
1951	Jan 15, [illegible]	[illegible]	[illegible]				

YOUNG CREEK NEAR EDINBURG, INDIANA



(25) Tippecanoe River at Oswego, Ind

Location. Lat 41°19'14", long 85°4[illegible]'1[illegible]", in NE¼ SE¼ sec 14, T. 33 N., R 6 E., on left bank 10 ft downstream from dam at Tippecanoe lake outlet in Oswego, 3 miles east of Leesburg.

Drainage area.--115 sq mi

Gage. - Nonrecording gage Oct 1, 1949 to Aug. 13, 1953; recording gage thereafter. Datum of gage is 830 00 ft above mean sea level datum of 1929

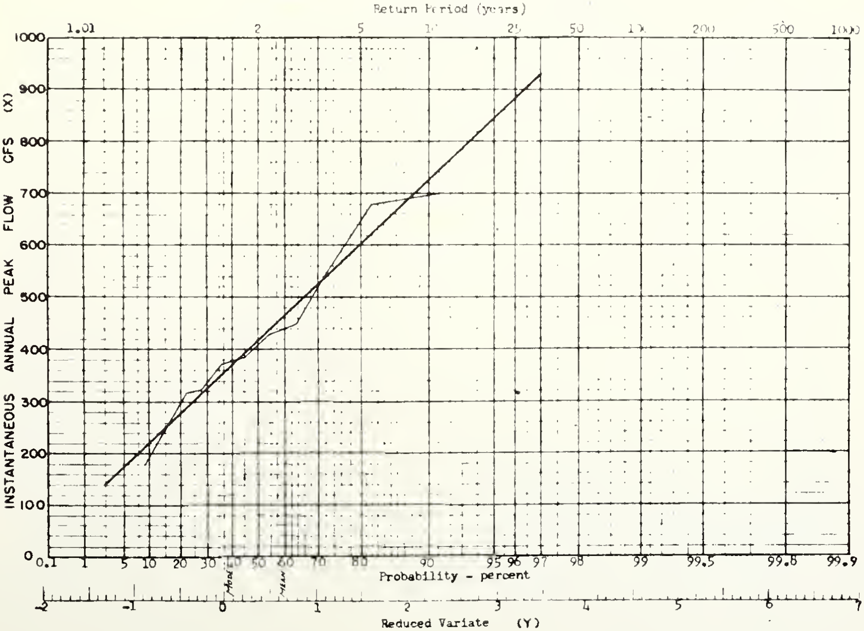
Stage-discharge relation.--Defined by current-meter measurements below 680 cfs and extended to 1,050 cfs by logarithmic plotting

Remarks. - Peak discharges affected by natural storage in numerous lakes upstream

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1943	May 23, 1943	9 4	1,050	1955	Oct 17, 1954	8 65	700
1950	Apr 8 10, 1950	8 62	[illegible]	1956	May 5 6, 1956	8 08	450
1951	Feb 27 28, 1951		430	1957	Apr 17 2[illegible],1957	7 59	315
1952	Jan 30, 1952		370	1958	Sept.1[illegible] [illegible],1957		383
1953	Mar 22,23, 1953	6 [illegible]	179	1959	[illegible] 18. 1959		548
1954	Apr 29,30, 1953	[illegible]	3[illegible]2				

TIPPECANOE RIVER AT OSWEGO, INDIANA



(26) North Fork Salt Creek near Belmont, Ind

Location --Lat 39°09'00", long 86°[illegible], in [illegible] sec 5, T 8 N, R 2 E, on right bank 15 ft downstream from bridge on State Highway 46, 100 ft upstream from Schooner Creek, 0 7 mile northeast of Belmont, 6 1/2 miles upstream from Brummett Creek, and 20 miles upstream from mouth

Drainage area 120 sq mi, includes that of Schooner Creek

Gage --Nonrecording gage Apr 4, 1946, to Oct 8, 1951; recording gage thereafter Altitude of gage is 546 ft (from topographic map)

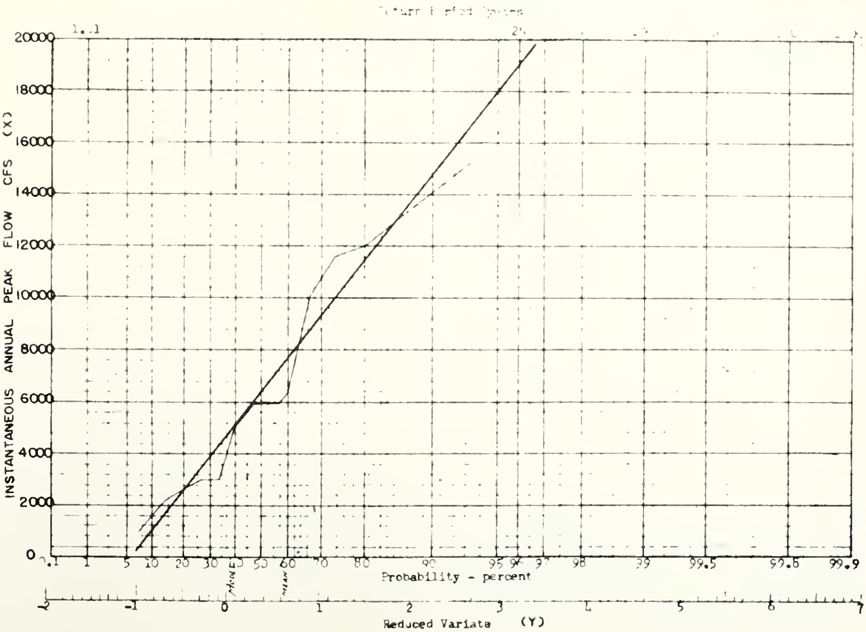
Stage-discharge relation --Defined by current meter measurements below 9,800 cfs Discharge adjusted for rate of change of stage above 7 ft Only annual maximums adjusted prior to installation of recording gage

Flood stage --16 ft

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1913	March 1913	25 7		1953	Mar 4, 1953	17 76	2,780
1946	May 16, 1946	20 3	5,910	1954	Jan 27, 1954	9 38	825
1947	June 2, 1947	21 2	10,100	1955	Mar 21, 1955	15 91	2,220
1948	Mar 27, 1948	18 0	3,010	1956	May 28, 1956	18 12	3,030
1949	Jan 5, 1949	22 2	13,300	1957	Apr 4, 1957	19 92	6,340
1950	Jan 4, 1950	21 7	11,600	1958	June 14, 1958		5,920
1951	Feb 21, 1951	19 53	5,100	1959	Jan 21, 1959		12,000
1952	May 24, 1952	22 55	15,200				

NORTH FORK SALT CREEK NEAR BELMONT, INDIANA



(27) Singleton ditch at Schneider, Ind.

Location.--Lat 41°12'44", long 87°26'44", on line between NE 1/4 sec. 21 and NW 1/4 sec. 22, T. 32 N., R. 9 W., on left bank 15 ft upstream from bridge on U. S Highway 41, half a mile upstream from Bruce ditch, 1 1/2 miles downstream from Cedar Creek, and 1 2/3 miles north of Schneider.

Drainage area.--122 sq mi.

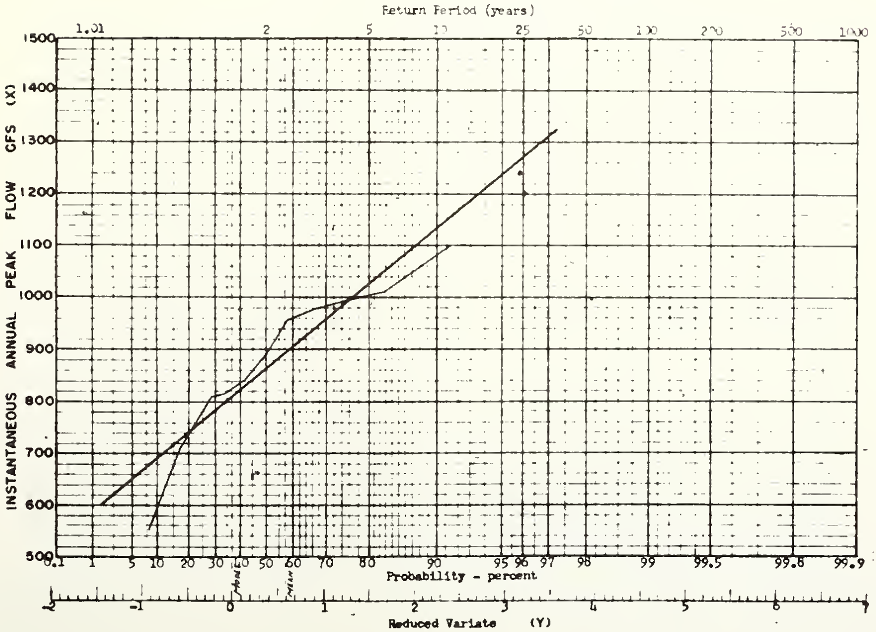
Gage.--Nonrecording gage July 28, 1948, to Aug. 13, 1951; recording gage thereafter. Prior to Oct. 1, 1949, at datum 2.00 ft higher. Datum of present gage is 623 67 ft above mean sea level, datum of 1929.

Stage-discharge relation.--Defined by current-meter measurements. Dredging in 1950 and subsequent floods and channel deterioration have materially affected the stage-discharge relation.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1949	Feb 15, 1949	-	550	1955	Oct. 11, 1954	10.10	953
1950	Apr 10, 1950	-	1,100	1956	Feb. 25, 1956	9 62	888
1951	Feb. 19, 1951	8.50	841	1957	Apr 28, 1957	10 27	979
1952	June 15, 1952	9.82	1,010	1958			714
1953		8.39	812	1959	Feb. 14, 1959		992
1954	Mar. 25, 1954	9.04	810				

SINGLETON DITCH AT SCHNEIDER, INDIANA



(28) East Fork Whitewater River at Richmond, Ind.

Location.--Lat 39°48'24", long 84°54'25", in SE 1/4 sec. 7, T. 13 N., R. 1 W., on left bank 50 ft downstream from highway bridge, three-quarters of a mile south of Richmond, and 2 miles upstream from Short Creek.

Drainage area.--123 sq mi.

Gage.--Nonrecording gage Apr. 27, 1949, to July 26, 1949; recording gage thereafter. Datum of gage is 854.01 ft above mean sea level, datum of 1929 (levels by Indiana Flood Control and Water Resources Commission).

Stage-discharge relation.--Defined by current meter measurements below 5,100 cfs and by slope-area measurement at 13,500 cfs.

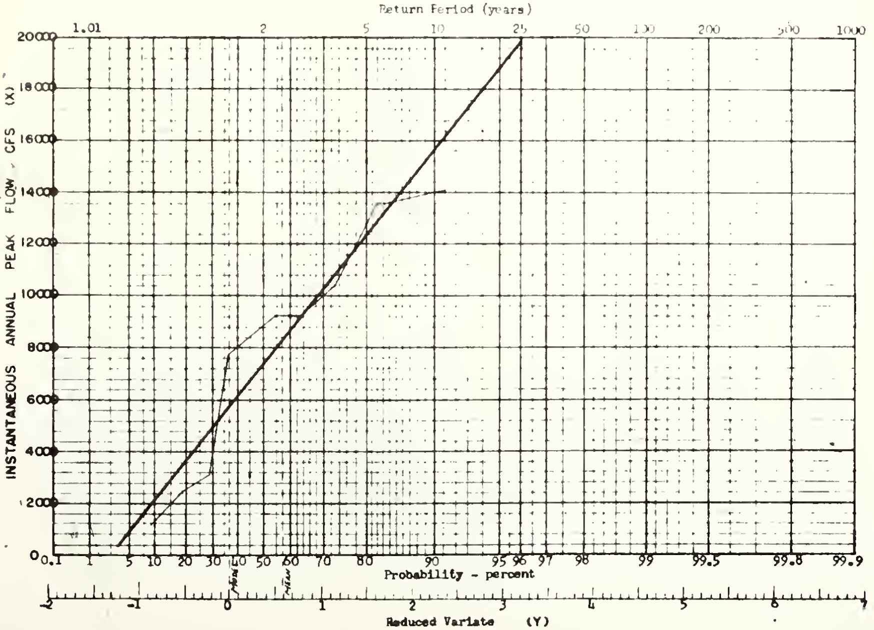
Flood stage.--10 ft.

Historical data.--Flood of September 1866 was reported by the Indianapolis Journal to be higher than ever before known. Flood of March 1913 is the maximum stage known according to information by local residents.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1913	March 1913	15.0	-	1955	Feb 21, 1955	6.07	2,540
1950	Jan 15, 1950	12.49	13,500	1956	Nov 16, 1955	10.70	8,200
1951	Nov 20, 1950	10.82	9,230	1957	June 28, 1957	10.54	7,800
1952	Jan. 26, 1952	10.66	9,250	1958	Aug. 2, 1958		10,400
1953	May 22, 1953	6.53	3,160	1959	Jan. 21, 1959		14,100
1954	Mar. 30, 1954	3.86	1,160				

EAST FORK WHITE RIVER AT RICHMOND, INDIANA



(29) Deep River at Lake George Outlet at Hobart, Ind.

Location.--Lat 41°32'10", long 87°15'25", in NW 1/4 sec. 32, T. 36 N., R. 7 W., on left bank at upstream side of highway bridge, 300 ft upstream from Duck Creek, and 400 ft downstream from Lake George Dam.

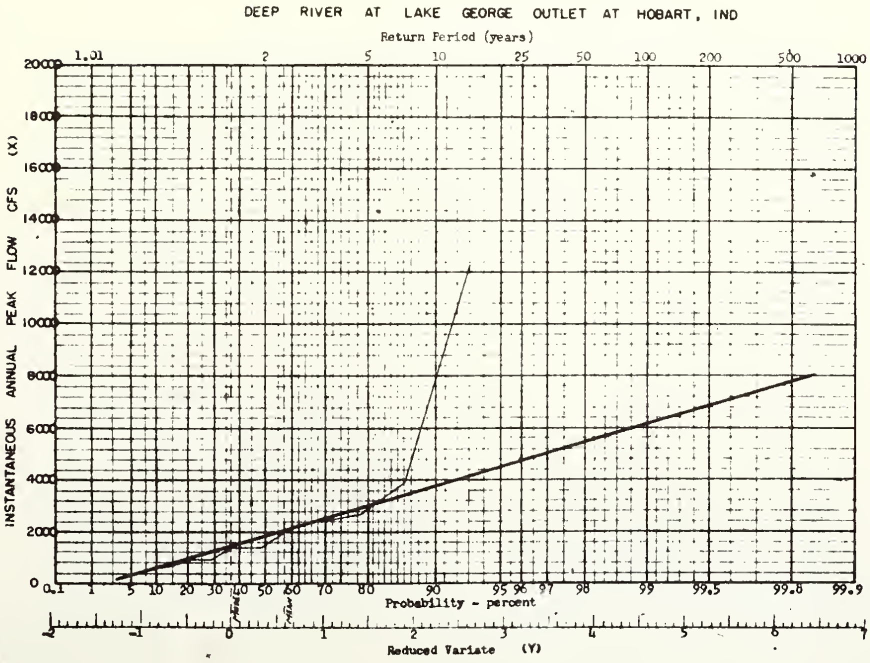
Drainage area.--125 sq mi.

Gage.--Nonrecording gage Apr. 4, 1947, to July 29, 1952; recording gage thereafter. Prior to July 21, 1955, at site 400 ft upstream at datum 11.80 ft higher than present datum. Datum of present gage is 588.17 ft above mean sea level, datum of 1929 (levels by Indiana Flood Control and Water Resources Commission).

Stage-discharge relation.--Defined by current-meter measurements below 3,300 cfs.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1947	Apr. 6, 1947	5.41	2,410	1954	Mar. 26, 1954	4.55	1,440
1948	May 11, 1948	5.86	2,740	1955	Oct 11, 1954	7.68	3,880
1949	Feb 14, 1949	3.50	620	1956	May 11, 1956	11.15	1,320
1950	Dec. 22, 1949	5.35	2,390	1957	July 14, 1957	12.35	1,650
1951	May 11, 1951	4.52	1,440	1958	June 10, 1958		720
1952	Nov. 14, 1951	4.41	1,340	1959	July 24, 1959		1,970
1953	Mar. 16, 1953	3.86	912				



(30) Big Ind[illegible]a [illegible]reek [illegible] [illegible]rydon, Ind

Location.--Lat 38°16'35", long 86°0[illegible]'[illegible]5", in S[illegible] 1/4 sec. 6, [illegible] 3 S., R. 4 E., on upstream side of bridge on Sta[illegible]e Highway 335, 0.6 mile upstream from Raccoon Branch and 4 1/2 miles north of [illegible]orydon.

Drainage area.--12[illegible] sq mi.

Gage.--Nonrecording gage Oct [illegible]2, 194[illegible], to Dec. 8, 1948; recording gage thereafter. [illegible]atum of gage is 577.12 ft above mean sea level, datum of 1929.

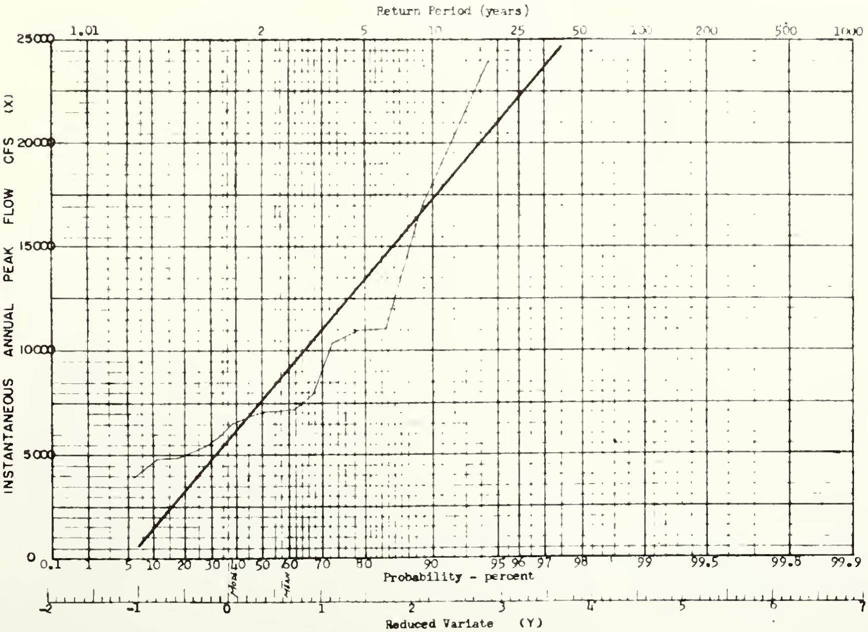
Stage-discharge relation.--Defined by current-meter measurements below 6,600 cfs and extended above [illegible] logarithmic plotting.

Historical data.--Flood of Mar. 1[illegible], 1[illegible]43, is the maximum known at Corydon since beginning of knowledge in 18[illegible]5.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage height	Discharge cfs	Water Year	Date	Gage height	Discharge cfs
194[illegible]	[illegible] 1[illegible], [illegible]	[illegible]	[illegible]	19[illegible]2	Mar. 11, 1952	1[illegible] 75	10,400
1944	Apr. 1[illegible], [illegible]	1[illegible]	4,[illegible]	19[illegible]3	Mar. 3, 1953	14 57	5,400
1945	[illegible], 19[illegible]	19.[illegible]	[illegible]	19[illegible]	Sept 30, 19[illegible]	1[illegible] 04	4,800
1946	Feb. [illegible], 194[illegible]	15.0	[illegible]	19[illegible]	Mar. 15, 1955	1[illegible] 98	3,900
1947	[illegible]	14 3	[illegible]	19[illegible]	Feb. [illegible]5, 1956	16 25	7,180
1948	[illegible] 1[illegible], [illegible]	19.[illegible]	[illegible]	[illegible]	[illegible], 1957	16.[illegible]2	7,300
1949	[illegible] 1[illegible], [illegible]	1[illegible].20	[illegible]	[illegible]	[illegible] 1[illegible], 1957		6,590
1950	[illegible] [illegible]9, 1950	16 [illegible]	7,[illegible]0	[illegible]	[illegible] 21, 1959		23,800
19[illegible]1	[illegible]	[illegible]	[illegible]				

BIG INDIANA CREEK NEAR LORYDON , INDIANA



(31) Mississinewa River near Ridgeville, Ind

Location - Lat [illegible]°17', long 85°00', in [illegible] 1/4 sec [illegible], T 19 N, R 14 E, on right bank 10 ft downstream from highway bridge, 0 8 mile downstream from Mud Creek, and 2 miles east of Ridgeville

Drainage area - 130 sq mi

Gage - Nonrecording gage Aug. 30, 1946 to Oct 3, 1950; recording gage thereafter Datum of gage is 965 23 ft above mean sea level, datum of 1929

Stage-discharge relation.- Defined by current meter measurements below 3,400 cfs

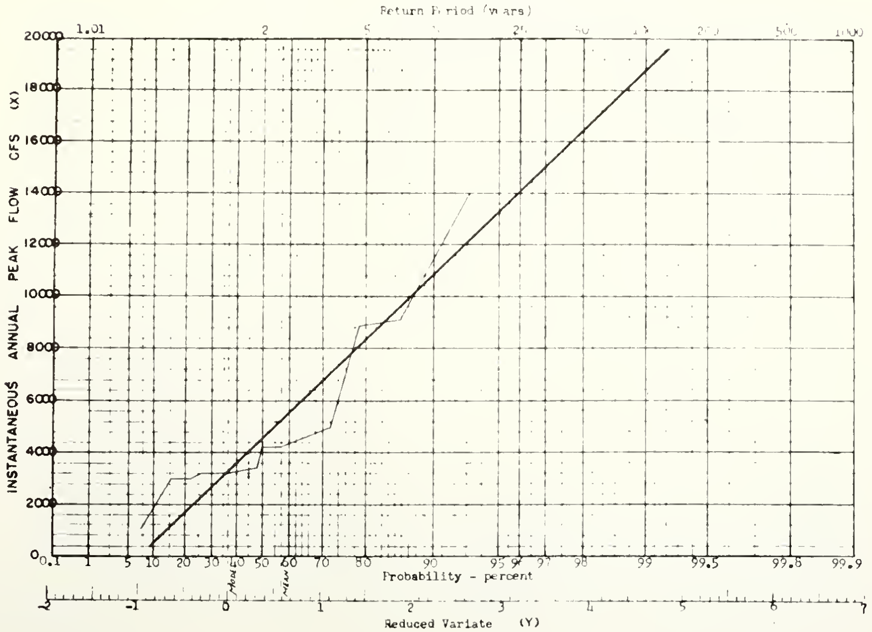
Flood stage.--10 ft

Historical data.--Local residents stated that the 1913 flood was secondary to a flood in the early 1930's when the river reached an estimated stage of 15 0 ft

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1947	Jan 30, 1947	11 16	2,490	1954	Mar 30, 1954	7 80	1,020
1948	Jan 1, 1948	12 2	3,480	1955	Jan. 6, 1955	11 16	2,490
1949	Jan 5, 1949	13 1	4,560	1956	Nov 16, 1955	12 79	4,200
1950	Feb. 14, 1950	13 4	4,920	1957	June 28, 1957	14.57	8,830
1951	Feb 21, 1951	12 75	4,200	1958	June 10, 1958		13,900
1952	Jan 26, 1952	11 [illegible]	3,250	1959	Jan 21, 1959		[illegible]
1953	Mar 4, 1953	12 00	3,250				

MISSISSINEWA RIVER NEAR RIDGEVILLE, INDIANA



(33) Kankakee River near North Liberty, Ind.

Location.--Lat 41° 3[illegible]'50", long 86°2[illegible]'[illegible]", on line between secs 11 and 23, T. 36 N., R. 1 W., on left bank at downstream side of bridge on St. Joseph County highway named "New Road," [illegible] miles northwest of North Liberty.

Drainage area.--15[illegible] sq mi.

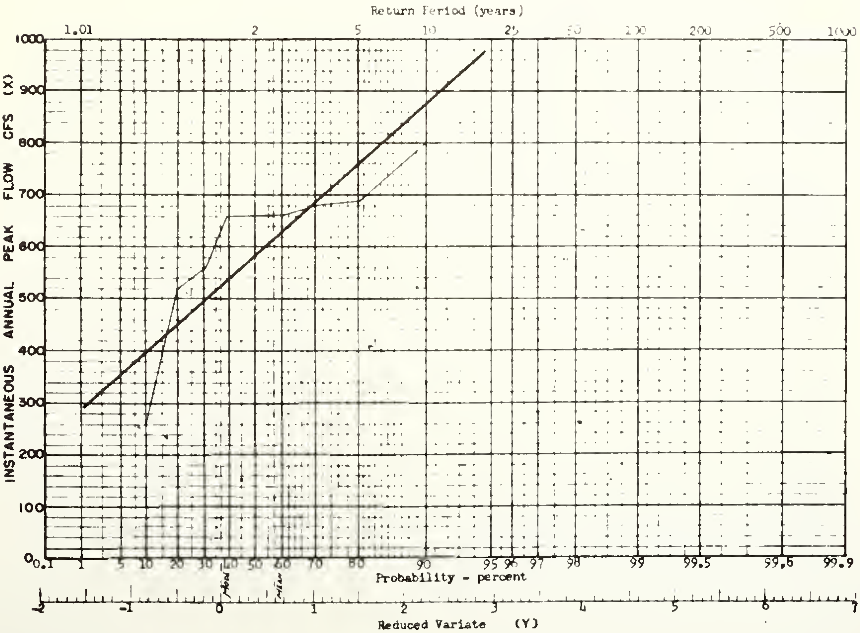
Gage.--Nonrecording gage Jan. 1[illegible], 1951, to June 2[illegible], 1[illegible]56; recording gage thereafter. Datum of gage is 680.04 ft above mean sea level, datum of 1929 (levels by Indiana Flood Control and Water Resources Commission).

Stage-discharge relation.--Relation affected by varying amount of backwater caused by return flow from overbank storage. Frequent current meter measurements necessary to define relationship during this period.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1951	May 11, 195[illegible]	6.25	[illegible]20	1956	Apr 30, 1956	6.92	660
1952	Nov. 14, 195[illegible]	[illegible].97	[illegible]	1957	[illegible] [illegible]7, 1957	6.90	660
1953	Mar. 1[illegible], 1953	4.4[illegible]	260	1958	Dec 20, 1957		560
1954	Apr 26, 1954		600	1959	Jan 27, 1959		560
1955	Oct [illegible], 1954	[illegible].64	686				

KANKAKEE RIVER NEAR NORTH LIBERTY, INDIANA



(34) Sand Creek near Brewersville, Ind.

Location --Lat 39°05'05", long 85°39'30", in NW¼ sec 5, T 7 N, R 8 E, on left bank at downstream side of county highway bridge, 2½ miles west of Brewersville, and 5 2 miles upstream from Bear Creek.

Drainage area -156 sq mi; 153 sq mi prior to Oct 6, 1952.

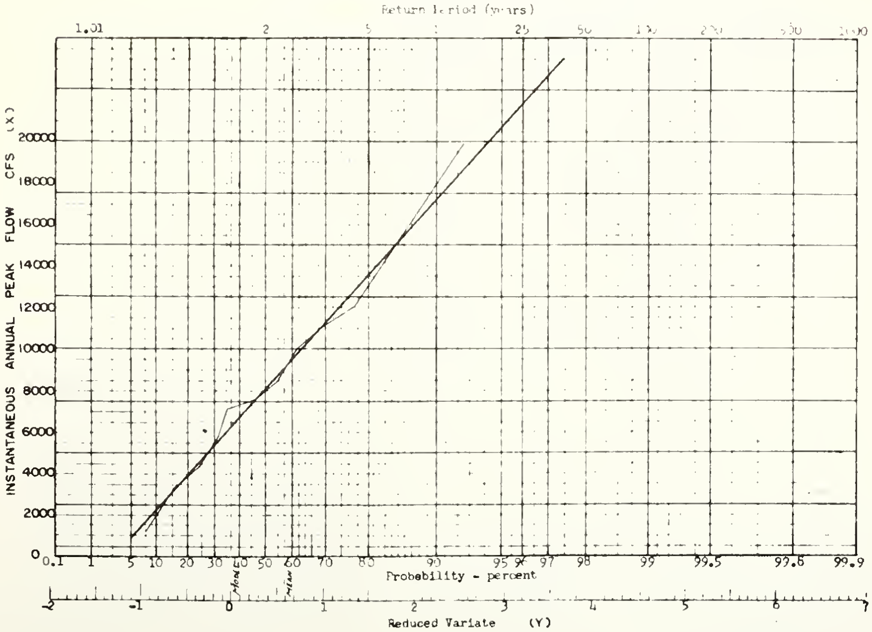
Gage.--Nonrecording gage Feb. 11, 1948, to Oct 5, 1952, at bridge 1.7 miles upstream at datum approximately 8 ft higher. Recording gage since Oct. 6, 1952, at present site Altitude of present gage is 630 ft (by altimeter).

Stage-discharge relation.--Defined by current-meter measurements at former site and by gage-height relationship with former site at present location

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1948	Mar 27, 1948	17 5	9,980	1954	June 1, 1954	5 75	1,240
1949	Jan 5, 1949	19 0	12,100	1955	Feb 27, 1955	11 42	4,300
1950	Jan 4, 1950	19 2	12,400	1956	May 28, 1956	15 45	7,560
1951	Nov 20, 1950	18 4	11,100	1957	Apr 4, 1957	16.33	8,480
1952	Jan 26, 1952	13 4	5,780	1958	July 22, 1958		7,150
1953	Mar 4, 1953	10 19	3,460	1959	Jan. 21, 1959		19,900

SAND CREEK NEAR BREWERSVILLE, INDIANA



Location Lat 40°27', long 85°57', on line between secs 9 and 10, T 23 N, R 5 E, on left bank at downstream side of bridge on State Highway 213, 1.5 miles south of Greentown

Drainage area 162 sq mi; 172 sq mi prior to June 5, 1954

Gage. -Nonrecording gage Feb. 20, 1945, to June 4, 1954; recording gage thereafter Prior to June 5, 1954, at site 1.0 miles downstream at datum 5.34 ft lower than present datum Datum of present gage is 804.35 ft above mean sea level, datum of 1929

Stage-discharge relation Defined by current meter measurements.

Flood stage 11 ft at both sites

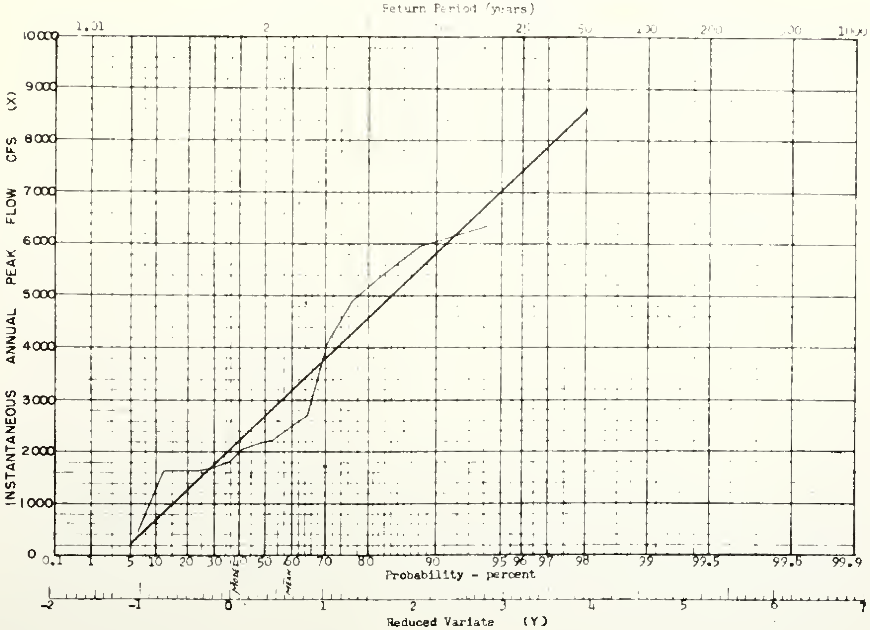
Historical data The following stories to appear in old newspapers for Kokomo, about 9 miles downstream September 1865: "Wildcat Creek raging; no train for 3 days " 1904: "Greatest flood in Kokomo history "

Flood of August 1871 reached a stage 3 inches below that of the 1913 flood at a bridge 1½ miles downstream from present site according to information by local resident on the basis of remembered high water marks made on the same tree

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1943	May, 1943	15 0	5 960	1952	Mar. 11, 1952	11 52	2,580
1945	Apr 1, 1945	10 04	1,680	1953	Mar 4, 1953	10 34	1.810
1946	Oct 2. 1945	9.94	1,640	1954	Apr 12, 1954	5 63	450
1947	Apr 30, 1947	10 94	2,140	1955	Jan 7, 1955	10 00	1 650
1948	Mar 22, 1948	11 63	2,670	1956	May 28, 1956	9 97	1,650
1949	Jan 19 1949	1 19	4,110	1957	June 29, 1957	12 17	2,260
1950	Jan 4 1950	15 3	6,120	1958	June 10, 1958		4,900
1951	Feb 21 1951	10 7	2,020	1959	Feb 10 1959		5,390

WILDCAT RIVER NEAR GREENTOWN, INDIANA



(36) Fall Cree[illegible] ne[illegible] Fortville, Ind

Location Lat 39° [illegible]7 15", long 85°5[illegible], in sec 5, T 17 [illegible] R 6 E, on right bank [illegible] downstream side of bridge on State Highway 238, 1 mile downstream from Lick [illegible] and 2 miles northwest of Fortville

Drainage area - 172 sq mi

Gage.--Nonrecording gage July 1, 1941 to June 26, 1942; recording gage thereafter Datum of gage is 787 43 ft above mean sea level, datum of 1929 (levels by Indianapolis Water Co)

Stage discharge relation --Defined by current meter measurements

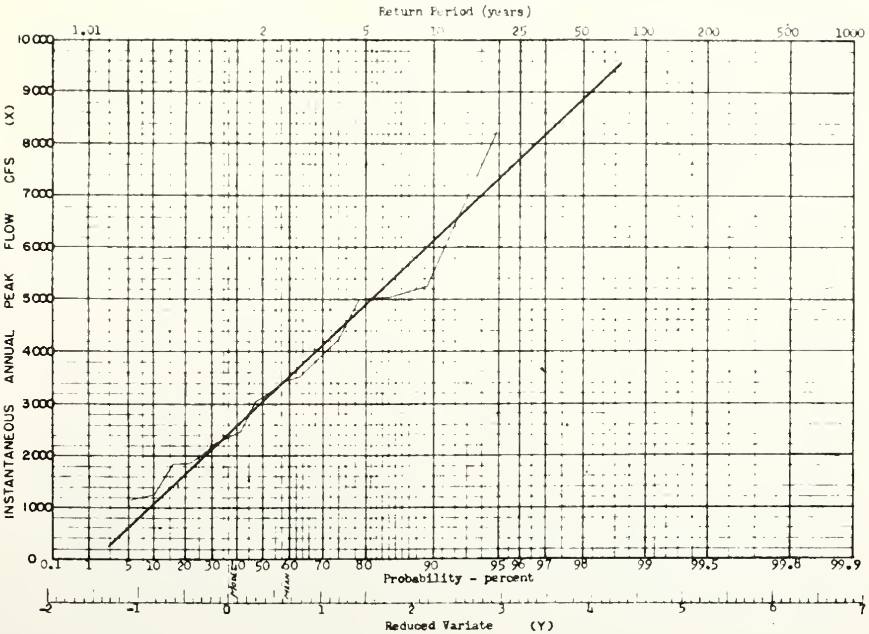
Flood stage [illegible] ft

Historical data.--Flood of 1913 reached a stage of about 12 feet according to information from local resident.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1942	Mar 17, 1942	7 39	2,460	1951	Feb 22, 1951	8 36	4,250
1943	May 18, 1943	9 77	8,240	1952	Jan. 27, 1952	6,85	2 000
1944	Apr 1[illegible] 1944	8 79	5,000	1953	July 6, 1953	8 14	3,850
1945	June 17, 1945	6 86	1,840	1954	Mar 30, 1954	5 50	1,140
1946	Oct 2, 1945	6 87	1,840	1955	Jan 6, 1955	5 76	1,260
1947	July 15, 1947	7 [illegible]	2,[illegible]0	1956	Feb 28, 1956	7 93	3,130
1948	Mar 24, 1948	7 97	3,500	1957	June 29, 1957	8.12	3,430
1949	Jan 1[illegible], 1949	[illegible]	[illegible]	1958	June 14 1958		5,040
1950	Jan [illegible], 1950	[illegible]	[illegible]	[illegible]	[illegible] 1959		3,010

FALL CREEK NEAR FORTVILLE, INDIANA



(37)

Location.--Lat 39°40'40", long 86°15' [illegible], in [illegible] sec. 6, T. 15 N., R. 3 E., on right bank at downstream side of bridge on Lynnhurst Drive, 3.0 miles upstream from Little Eagle Creek, 5.0 miles west of Monument Circle in Indianapolis, and 6.7 miles upstream from mouth.

Drainage area.- 179 sq mi.

Gage.--Nonrecording gage Nov. 18, 1938, to [illegible] 19, 1939; recording gage thereafter. Datum of gage is 706.21 ft above mean sea level, datum of 1929.

Stage discharge relation.--Defined by current meter measurements below 9,000 cfs and extended above on basis of a combined current meter measurement and slope-area measurement. High-water relation has shown a tendency to shift. Discharge shown for the 1913 flood is an approximate value based on logarithmic extension of an early rating curve above [illegible] cfs.

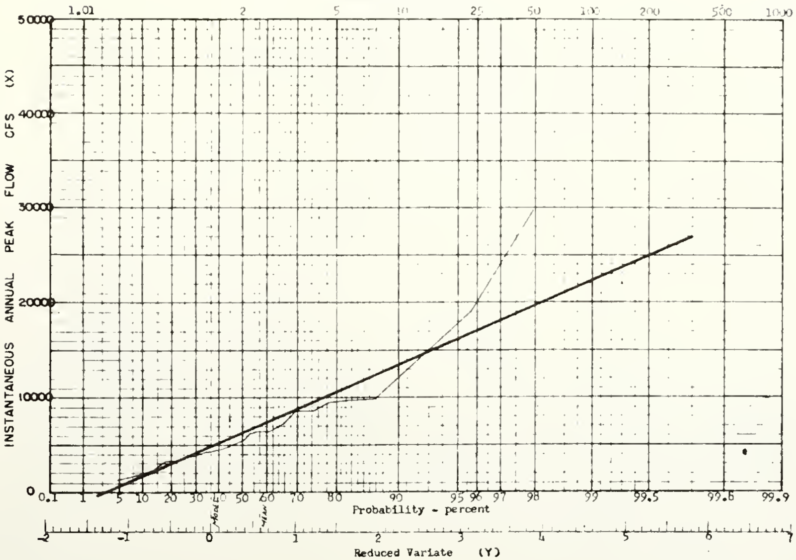
Historical data.-The following information was obtained from a report on "Eagle Creek at Indianapolis, Channel improvements for Flood Control" by Indiana Flood Control and Water Resources Commission, dated February 1956: "Investigations on past flooding by searching newspaper files, interrogation of local persons, examination of old surveys and reports, indicate that flooding occurred along Eagle Creek in 1875, 1904, 1913, 1915, 1926, 1933, 1943, and 1950. Newspaper accounts of flooding on other streams in the Indianapolis area indicate that flooding probably also occurred in 1847, 1858, 1866, 1882 and 1883."

"It is probable that the floods of 1875 and 1904 were among the greater floods on the stream. Newspaper accounts and weather records indicate that the flood of July 1915 was nearly as great as that of March 1913."

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1913*	March 1913	16.0	19,000	1947	Jan. 30, 1947	8.47	3,370
1938	April 1938	14.5	-	1948	Apr. 6, 1948	12.26	9,550
1939	Mar. 12, 1939	10.6	6,610	1949	Jan. 19, 1949	11.86	7,250
1940	Mar. 3, 1940	6.30	1,860	1950	Jan. 4, 1950	13.03	8,670
1941	June 12, 1941	5.77	1,470	1951	Feb. 21, 1951	8.47	3,950
1942	Feb. 7, 1942	8.66	4,120	1952	Jan. 27, 1952	9.88	5,520
1943	May 11, 1943	12.17	9,660	1953	[illegible], 1953	9.34	4,920
1944	Apr. 11, 1944	10.61	[illegible]	1954	Apr. 6, 1954	[illegible]	2,250
1945	Mar. 31, 1945	[illegible]	[illegible]	1955	July [illegible], 1955	6.9[illegible]	2,650
1946	May 17, 1946	8.99	3,800	1956	May 23, 1956	13.62	9,920
				1957	June 23, 1957	16.38	18,800
				1958	Aug. 9, 1958		8,560
				1959	Jan. 22, 1959		6,290

EAGLE CREEK AT INDIANAPOLIS, INDIANA



(38) Blue River at Carthage, Ind

Location Lat 39°46′, long 85°31′, in sec 18, T 15 N, R 9 E, on right bank 500 ft upstream from highway bridge, half a mile west of Carthage, and 2 ½ miles downstream from Three Mile Creek

Drainage area.- 187 sq mi

Gage.--Nonrecording gage Oct 11, 1950, to July 18, 1951; recording gage thereafter Prior to July 19, 1951, at bridge 500 ft downstream Datum of gage is 859 33 ft above mean sea level, datum of 1929.

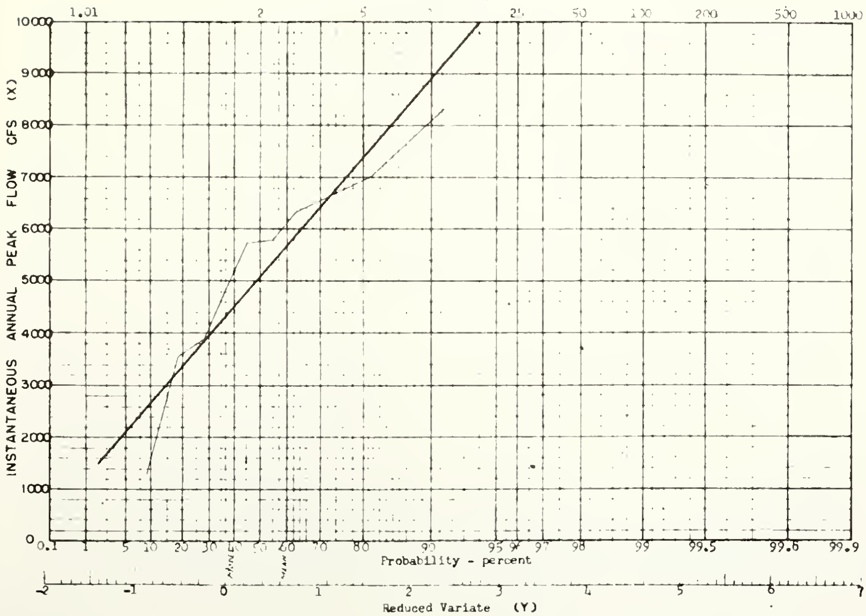
Stage-discharge relation.--Defined by current-meter measurements

Flood stage 7 ft

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1949	Jan 5, 1949	10 6	5,750	1955	Jan 6, 1955	5 97	1,290
1951	Feb 21, 1951	11 2	6,650	1956	Nov 16, 1955	11.52	5,800
1952	Jan. 27, 1952	11 02	6,350	1957	June 18, 1957	9 77	3,900
1953	Mar 4, 1953	9 17	3,580	1958	June 14, 1958		7,020
1954	Apr 5, 1954	10 02	4,850	1959	Jan 21, 1959		8,340

BLUE RIVER AT CARTHAGE, INDIANA



(39) Silver Creek near Sellersburg

Location -- Lat [illegible]°22'[illegible], long 85 [illegible] [illegible]', in [illegible] Lot 58, Clark Military Grant, on upstream side of Straws Mill Bridge on [illegible]son Road, 0.3 mile downstream from Pleasant Run, 2.4 miles southeast of Sellersburg, and [illegible] miles upstream from mouth.

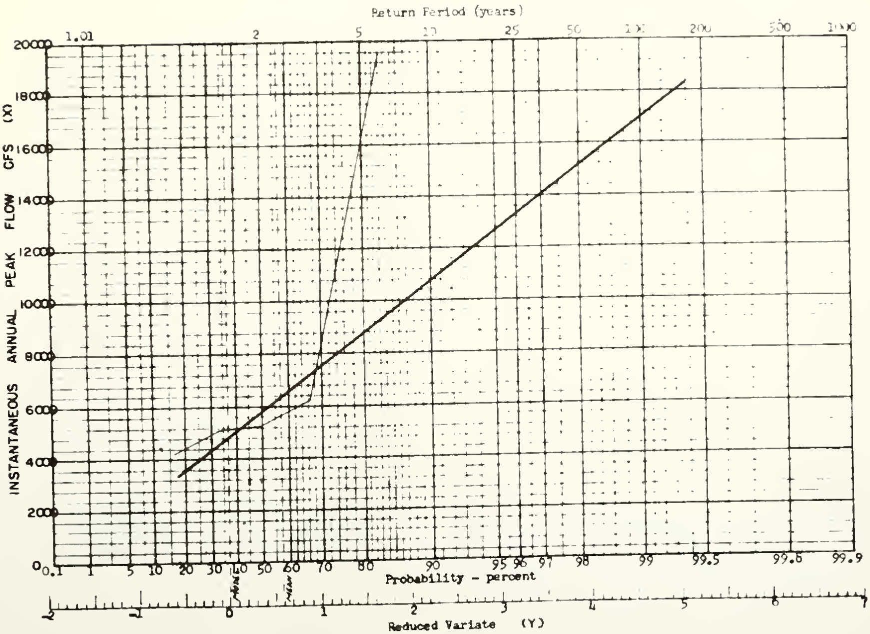
Drainage area -- [illegible] sq mi.

Gage.--Wire-weight gage read twice daily.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1955	Feb 22, 1955		4,320	1958	May 19, 1957		5,080
1956	Feb 2, 1956		5,350	1959	Jan 22, 1959		19,600
1957	May 2[illegible], 1957		6,250				

SILVER CREEK NEAR SELLERSBURG, INDIANA



(40) Busseron Creek near Carlisle, Ind

Location.--Lat 38°58 30", long 87°25 35", in W½ sec 17, T 6 N., R 9 W., on right bank 10 ft downstream from bridge on State Highway 58, 1½ miles northwest of Carlisle, and 6 3/4 miles upstream from mouth.

Drainage area.--228 sq mi.

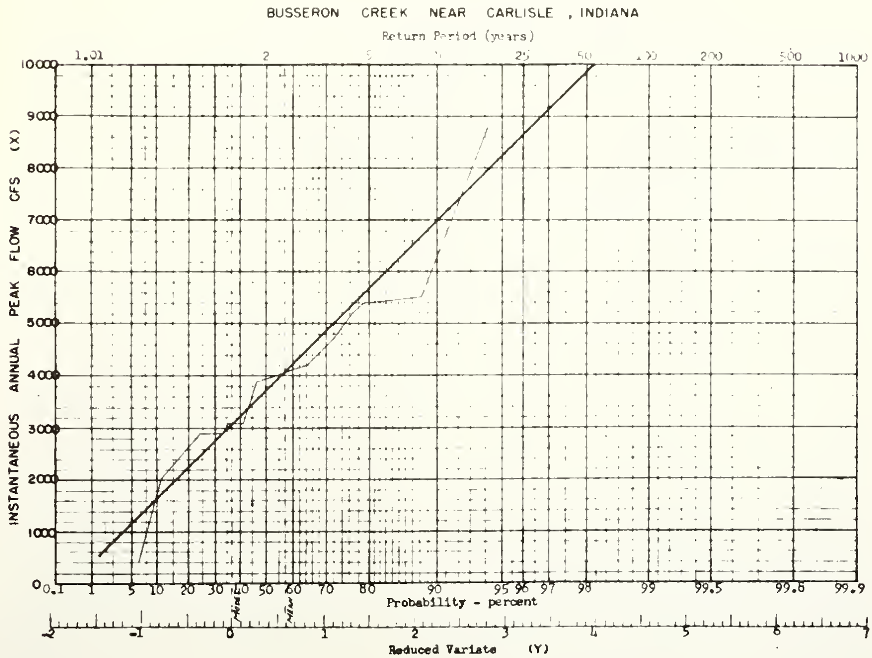
Gage.--Nonrecording gage Oct. 15, 1943, to Nov. 7, 1950; recording gage thereafter Datum of gage is 425.36 ft above mean sea level (State Highway Department of Indiana bench mark)

Stage-discharge relation.--Defined by current-meter measurements below 4,500 cfs and extended above by logarithmic plotting.

Flood stage.--12 ft.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1944	Apr 12, 1944	16 96	4,700	1952	Mar 11, 1952	16.17	4,070
1945	Apr 2, 1945	17 60	5,500	1953	Mar 4, 1953	16.04	3,890
1946	May 20, 1946	14 90	2,900	1954	Aug 4, 1954	6.31	430
1947	June 2, 1947	14 60	2,720	1955	Apr 13, 1955	13 13	2,040
1948	Jan. 3, 1948	15.15	3,100	1956	June 22, 1956	16.12	3,980
1949	Jan. 20, 1949	16.3	4,200	1957	May 23, 1957	17.61	5,200
1950	Jan. 5, 1950	20 05	8,800	1958	Dec 21, 1957		5,400
1951	Feb 21, 1951	14.75	2,900	1959	Jan. 22, 1959		3,100



Location.--Lat 38°57'05", long 85°04'22", in sec. 2, T. 4 N., R. 3 W., on right bank 2 miles southeast of Farmers Retreat and 3 3/4 miles downstream from Bear Creek.

Drainage area.--248 sq mi.

Gage.--Nonrecording gage Oct. 3, 1940, to Apr. 15, 1941; recording gage thereafter. Altitude of gage is 526 ft (by barometer).

Stage-discharge relation.--Defined by current-meter measurements below 14,000 cfs.

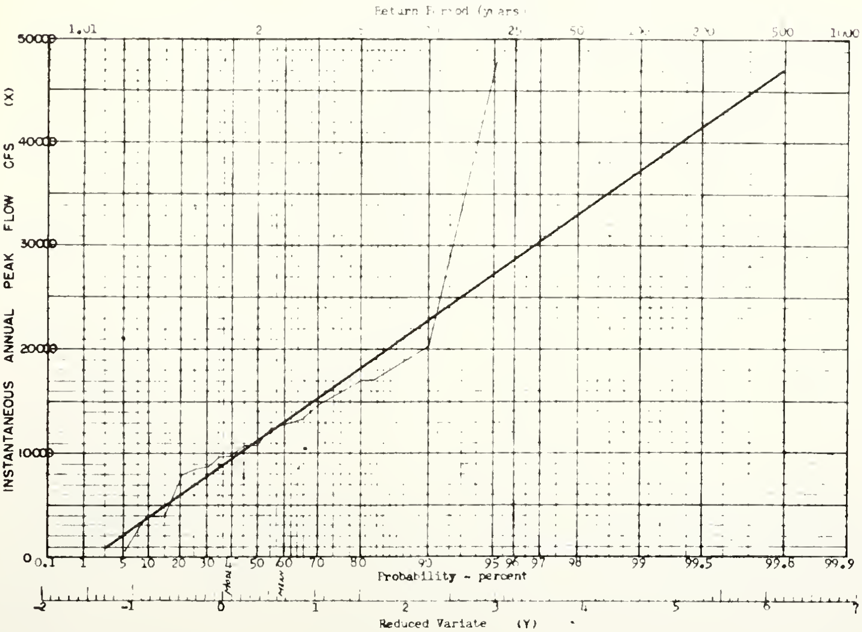
Flood stage.- 13 ft.

Historical data.--Flood of 1897 reached a stage of about 18 feet and is the highest known flood, from information by local residents.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1941	June 9 or 10, 1941	9.62	[illegible],960	1951	Jan. 3, 1951	13.50	9,660
1942	Apr 9, 1942	13.91	10,800	1952	Mar 10, 1952	13.46	9,660
1943	Mar. 19, 1943	14.50	12,900	1953	May 17, 1953	9.44	3,960
1944	Apr 11, 1944	13.16	8,880	1954	May 3, 1954	3.99	640
1945	Mar 6, 1945	15.54	17,000	1955	Mar 21, 1955	13.88	10,800
1946	Feb 13, 1946	12.7[illegible]	7,980	1956	May 28, 1956	14.45	12,500
1947	May 25, 1947	14.62	13,300	1957	July 5, 1957	16.15	20,200
1948	Apr 12, 1948	13.01	8,410	1958	July 22, 1958		17,000
1949	Jan 24, 1949	15.23	15,700	1959	Jan 21, 1959		47,800
1950	Feb 3, 1950	15.03	1[illegible],8[illegible]				

LAUGHERY CREEK NEAR FARMARS RETREAT, INDIANA



(42) Patoka River at Jasper, Ind

Location.--Lat 38°24'49", long 86°52'?0", in SE¼ sec 20, T 1 S., R 4 W., on left bank, 0.3 mile upstream from unnamed outlet of Jasper Lake, 1.0 mile downstream from Coon Seitz bridge, 1.2 miles downstream from Beaver Creek, and 3.3 miles northeast of Jasper.

Drainage area.--257 sq mi; 270 sq mi at former site.

Gage.--Nonrecording gage Nov 20, 1947, to Sept 17, 1956; recording gage thereafter. Prior to Sept 18, 1956, at site 5.6 miles downstream at datum 0.34 ft lower; datum of present gage is 446.19 ft above mean sea level, datum of 1929.

Stage-discharge relation.--Defined by current-meter measurements below 5,000 cfs at former site and below 1,100 cfs for present site.

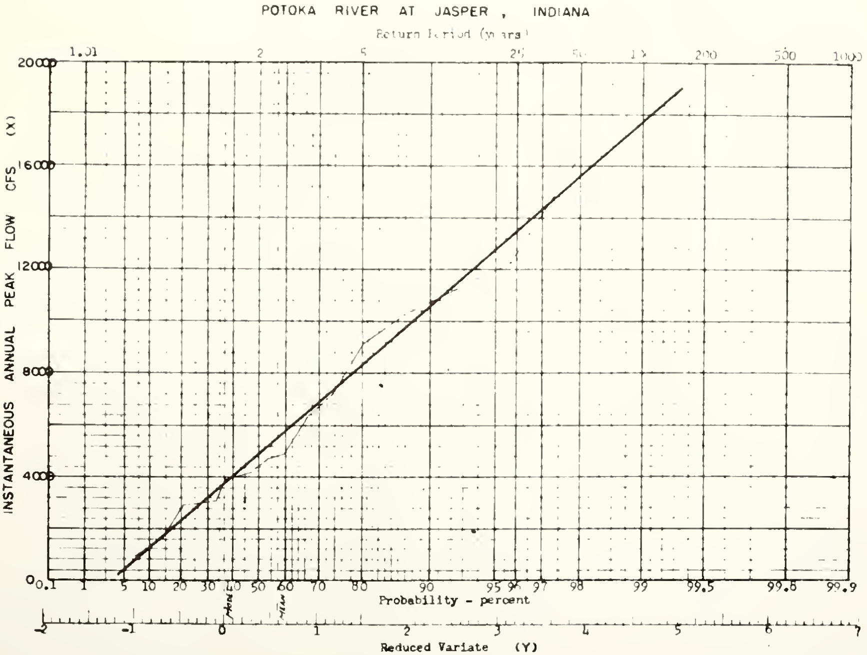
Flood stage.--14 ft; 9 ft at former site.

Historical data.--Flood of March 1913 is maximum stage known. Maximum stage at present site for period 1925-57, 20 ft in 1925 (information from local resident).

Remarks.--Flow slightly regulated by Beaver Creek Reservoir, whose outlet enters the Patoka River 1.2 miles upstream from the gage; peak discharges not materially affected.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1913	March 1913	15 9	16,000	1953	Mar 8, 1953	7 90	1,640
1937	January 1937	14 8	12,100	1954	Mar 2, 1954	-	950
1948	Apr 15, 1948	11 57	4,920	1955	Mar 3, 1955	9 80	2,940
1949	Jan. 28, 1949	11.13	4,220	1956	Feb 29, 1956	9.98	3,100
1950	Jan 7, 1950	12.37	6,300	1957	May 25, 1957	17 87	6,900
1951	Mar 21, 1951	11 46	4,760	1958	Dec 22, 1957		4,250
1952	Mar. 14, 1952	10 78	3,880	1959	Jan 24, 1959		9,150



APPENDIX A

Results of frequency analysis of the 32 watersheds studied with gaging station information

Appendix B - List of Symbols

A - area of watershed (sq. miles unless otherwise noted)

a - waterway area of culvert (sq. ft.); partial area; a coefficient; an exponent

b - an exponent

C - a coefficient

c - a coefficient; an exponent

D - drainage density (miles/sq. mile)

d - a coefficient

e - base of natural logarithms, $e = 2.72$

F - shape factor $F = L/(4A/\pi)^{1/2}$

H - mean relief of watershed (ft)

h - elevation above gaging station (ft)

i - a variable integer in summation operation

K - parameter in equation for instantaneous unit hydrograph (hours)

k - total number of entries in summation operation

K_1 - the recession constant of hydrograph (hours)

L - length of main stream in watershed (miles)

m - rank of entry in frequency analysis

N - total number of entries in summation operation

n - parameter of equation for instantaneous unit hydrograph and for hydrographs of short duration; an integer appearing in summation operation; total number of entries in extreme value series.

P - total rainfall depth during storm (inches)

P_L - rainfall depth occuring before start of runoff (inches)

P_x - rainfall depth occuring after start of runoff (inches)

P_i - rainfall during ith time interval (inches)

Q - discharge; direct surface runoff (cfs)

Q_B - base flow (cfs)

Q_m - annual peak discharge (cfs), peak discharge of the total runoff hydrograph.

Q_p - peak discharge of the direct surface runoff hydrograph

Q_T = total discharge; total runoff, $Q_T = Q + Q_B$, (cfs)

q = ordinate of the unit hydrograph (cfs/inch)

R = volume of direct surface runoff expressed in units of depth over watershed (inches)

r = runoff coefficient $r = R/I_2$

S = slope of main stream (ft/10,000 ft)

S_i = slope of ith section along main channel

s = an independent variable

T = duration of unit hydrograph (hours)

T_i = duration of unit hydrograph (hours)

T_r = return period (years)

t = time since [illegible] of direct surface runoff (hours); an independent variable

t_p = time to peak of unit hydrograph (hours)

U = ordinate of unit hydrograph of short duration (cfs/inch)

V = volume of direct surface runoff (acre ft)

x = an exponent; an independent variable; an extreme value

y = an exponent; a dependent variable

z = elevation above MSL (ft); an independent variable

z_o = elevation of gaging station (ft)

ϕ = cumulative probability

Γ = the gamma function

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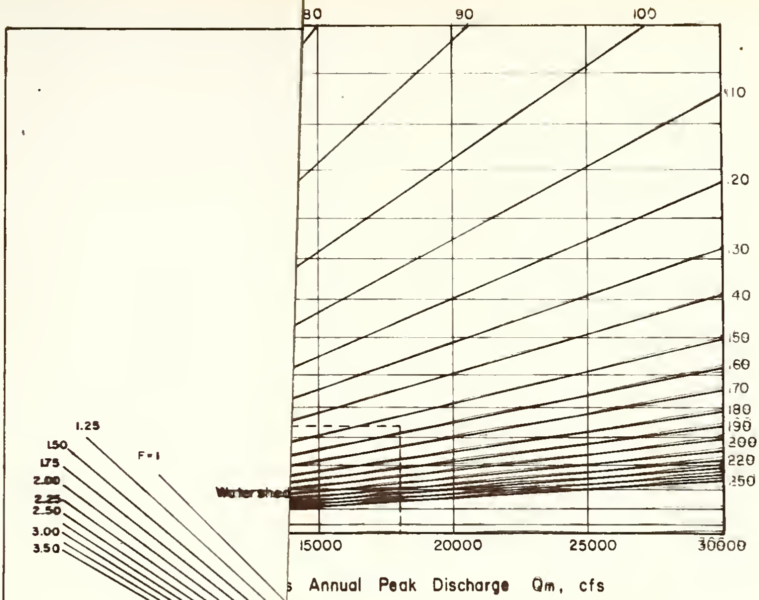
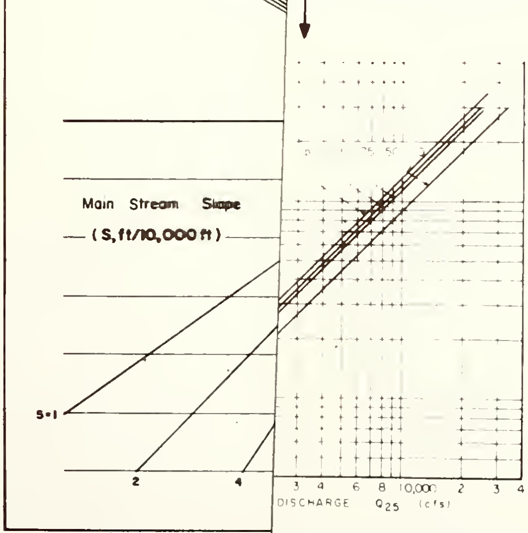


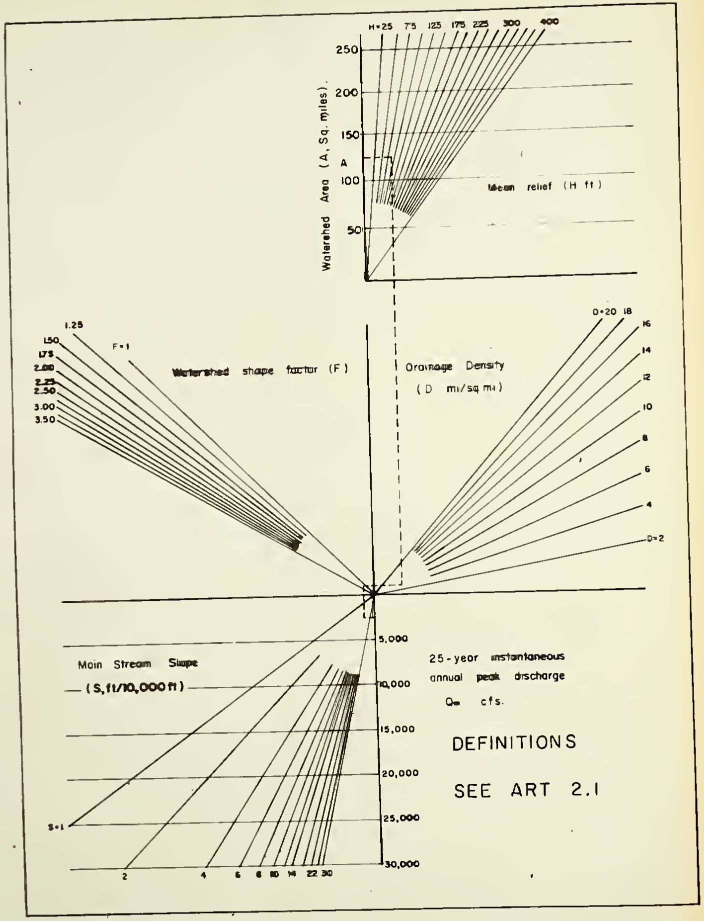
ARGE BY SIMPLE FORMULA (Eq. 4-1)



IARGE OF OTHER

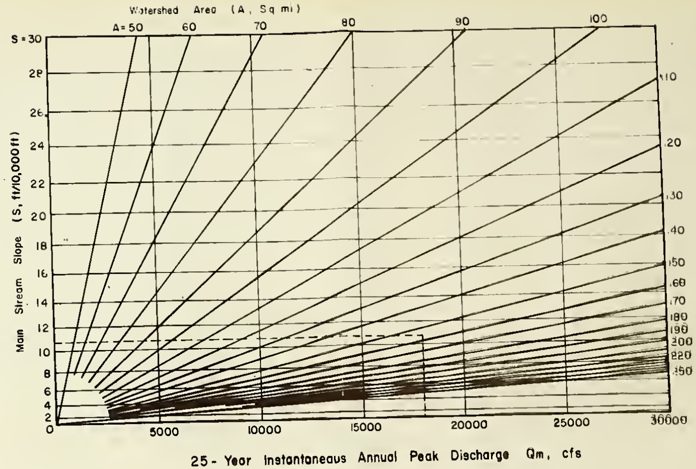
25 YEAR PEAKERIODS

A



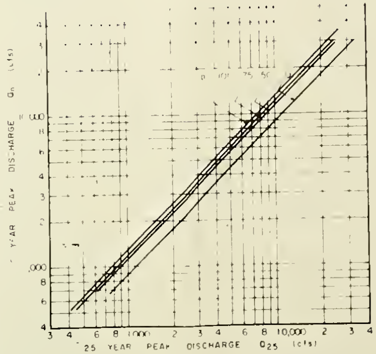
25 YEAR PEAK DISCHARGE BY EXTENDED FORMULA (Eq 4-2)

A'



25 YEAR PEAK DISCHARGE BY SIMPLE FORMULA (Eq. 4-1)

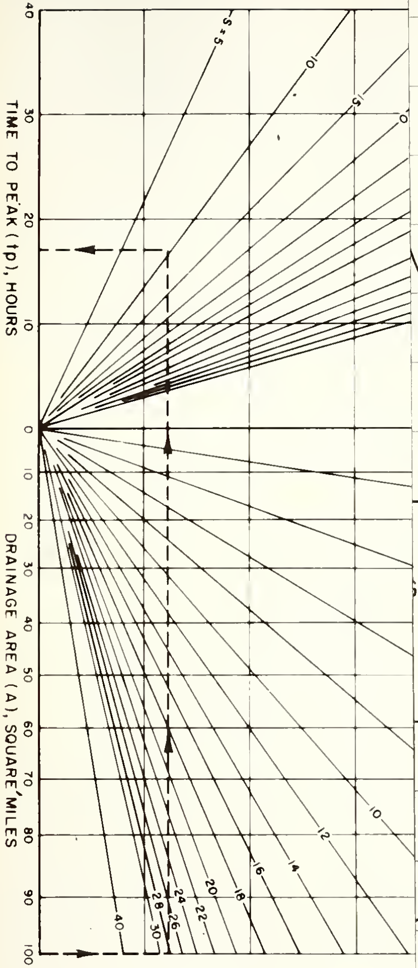
B



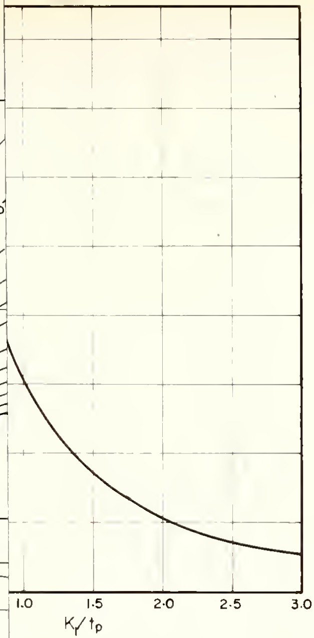
PEAK DISCHARGE OF OTHER RETURN PERIODS

DESIGN CHART NO 1

DETERMINATION OF ANNUAL PEAK DISCHARGE

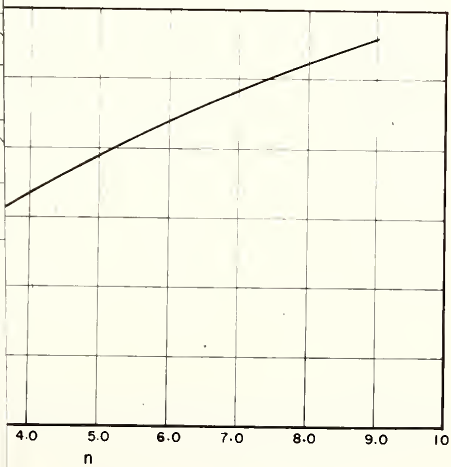


A



RAPH PARAMETER, n

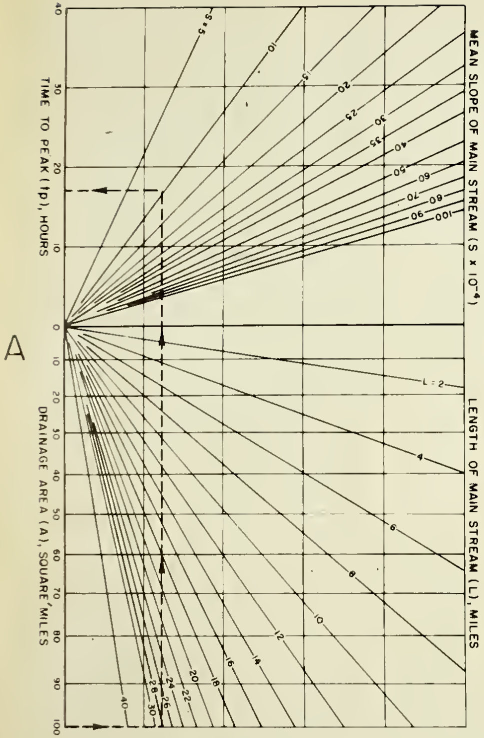
TIME TO PEAK, t_p



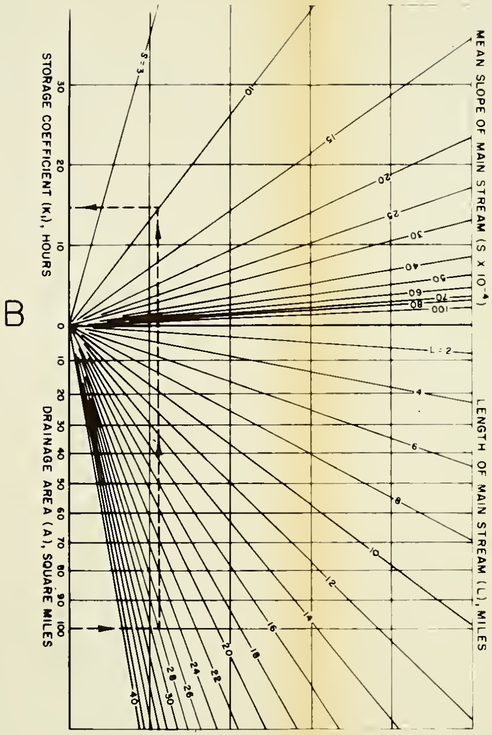
PEAK DISCHARGE Q_p

DESIGN

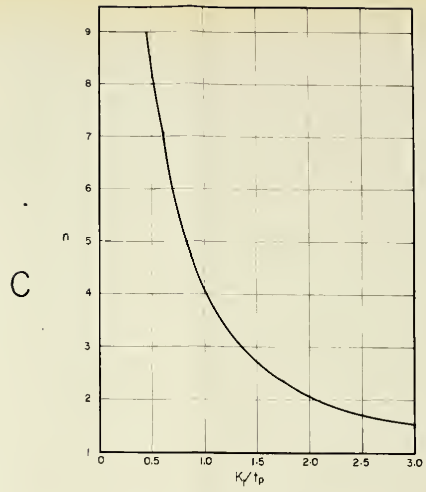
DETERM



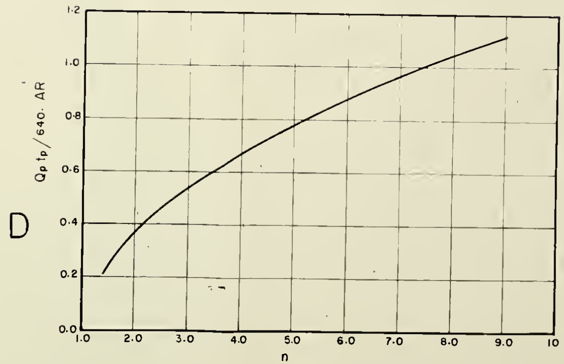
TIME TO PEAK, t_p



STORAGE COEFFICIENT, K_1



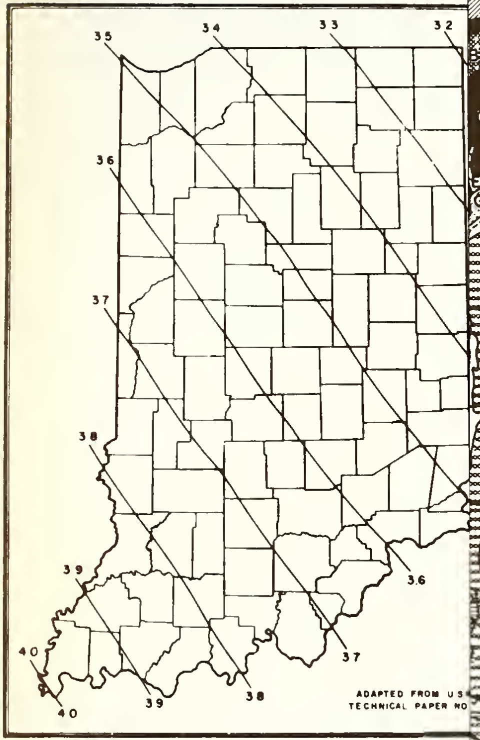
HYDROGRAPH PARAMETER, n



PEAK DISCHARGE Q_p

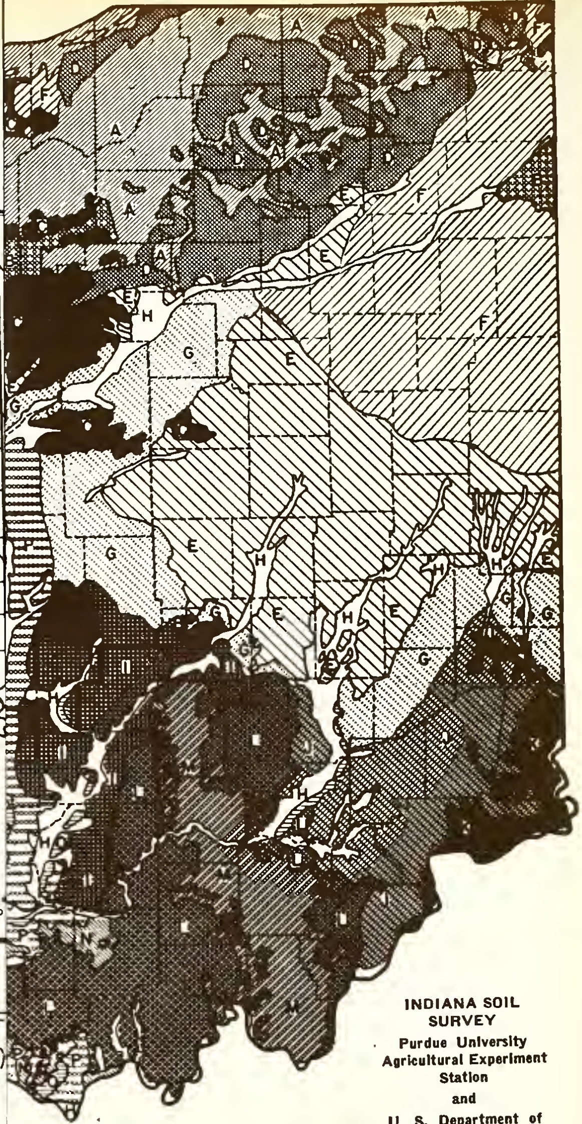
DESIGN CHART NO 2

DETERMINATION OF HYDROGRAPH OF SHORT DURATION



25 YEAR SIX HOUR RAINF

(in inches)



INDIANA SOIL SURVEY
Purdue University
Agricultural Experiment Station
and
U. S. Department of Agriculture

DESIGN CHART N

DETERMINATION

RUN OFF COEFFICIENT

0.30

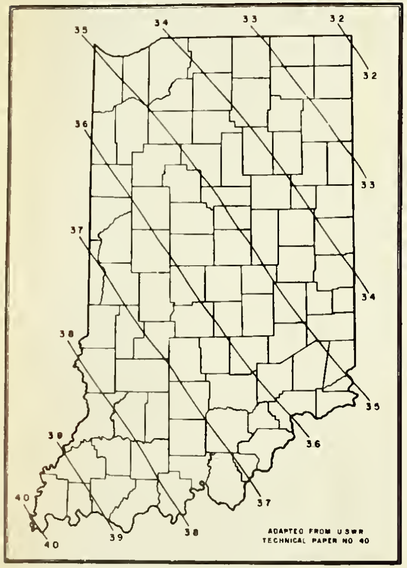
0.50

0.70

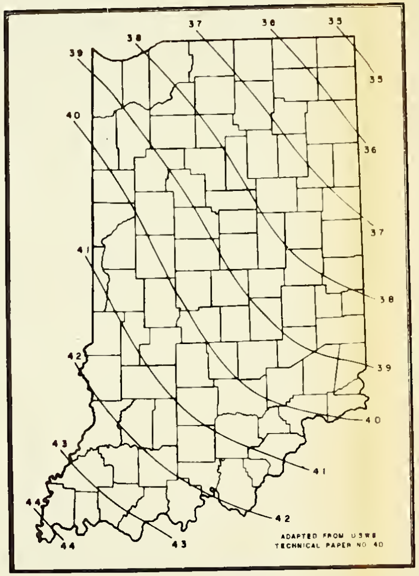
0.80

1.00

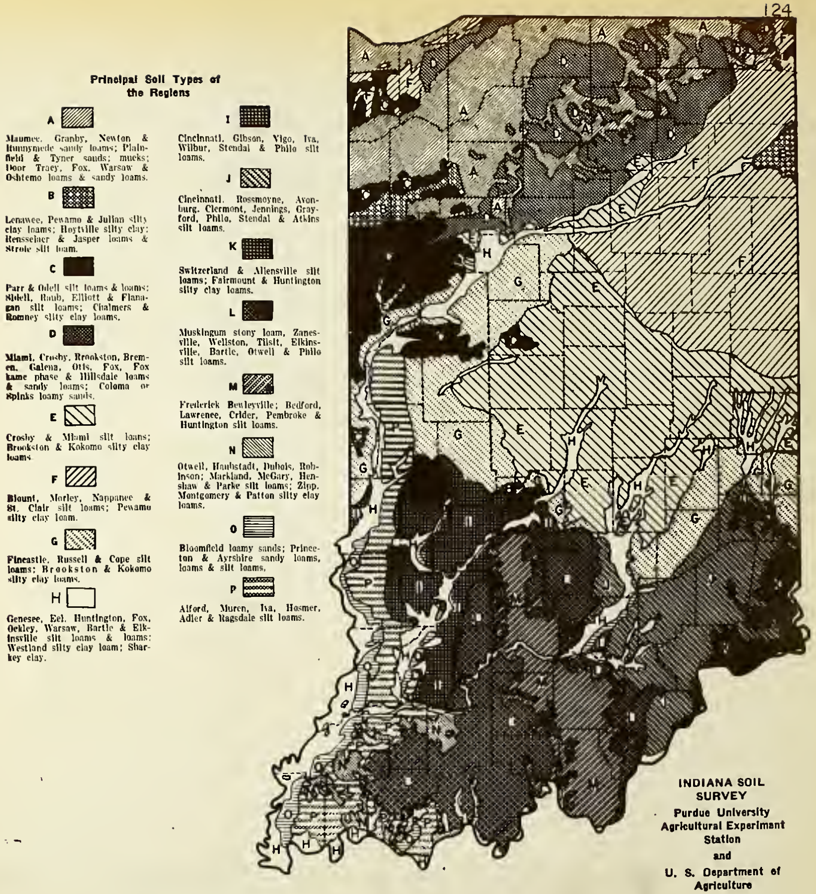
0.5 - 0.8



25 YEAR SIX HOUR RAINFALL
(in inches)



50 YEAR SIX HOUR RAINFALL
(in inches)



DESIGN CHART NO 3

DETERMINATION OF RAINFALL EXCESS

SOIL TYPE	RUN OFF COEFFICIENT
A,	0.30
D,H,O	0.50
C,E,G,M,P	0.70
K,L,N	0.80
B,I,J	1.00
F	0.5 - 0.8